CIVIL ENGINEERING PRACTICE

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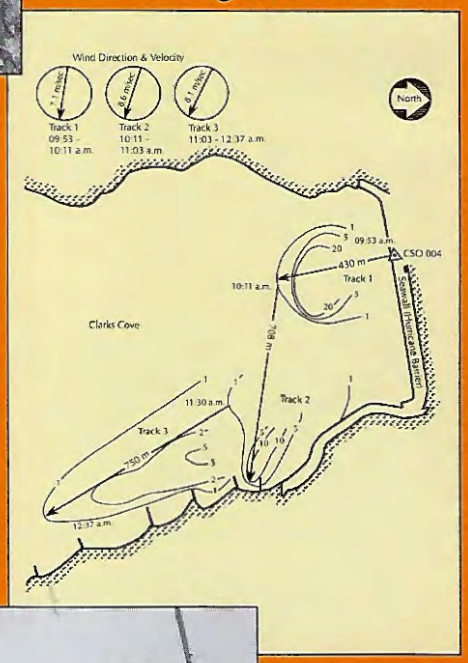
Tunnel Boring Machine Excavation

Mixing Zone for CSOs



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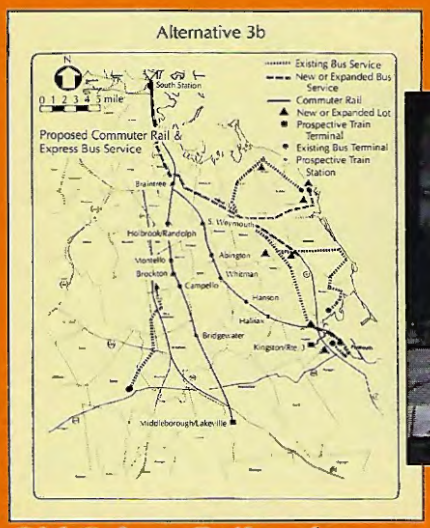

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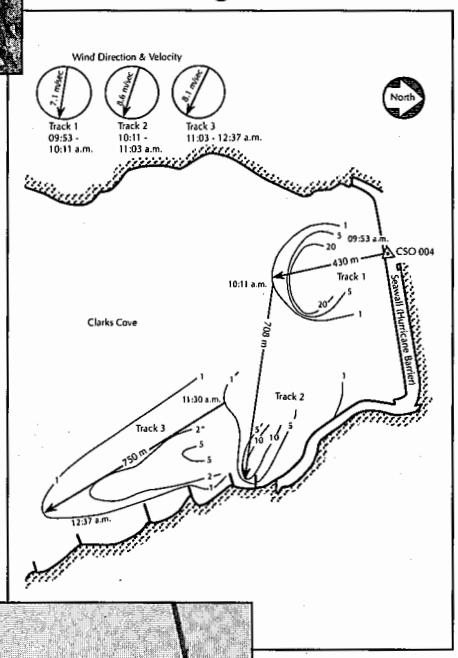

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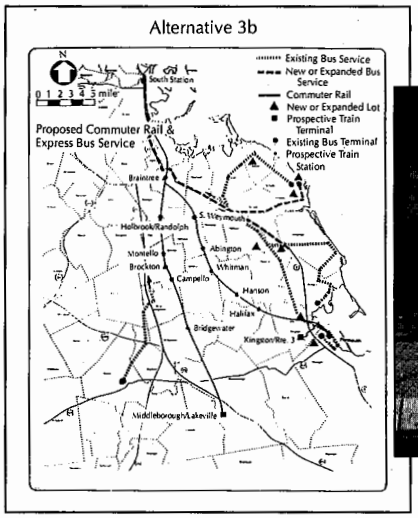

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Civil Engineering Practice
Boston Society of Civil Engineers Section/ASCE
The Engineering Center
One Walnut St.
Boston, MA 02108

Phone: (617) 227-5551
Fax: (617) 227-6783

The New York City Water Supply: Past, Present & Future

Meeting increasing demand, promoting conservation methods and complying with new water quality standards pose special challenges for major urban water supply systems.

EDWARD C. SCHEADER

The existing water supply system of the city of New York is derived from a watershed area of approximately 1,950 square miles, an area larger than the entire state of Delaware. The system consists primarily of three upland watershed supply areas: the Croton, the Catskill and the Delaware watersheds (see Figure 1).

The oldest of these three supplies, the Croton, was constructed over a period of 70 years, spanning the last six decades of the 19th century and the first decade of the 20th. This period was a time of tremendous growth not only in New York City but also in the nation as a whole. A vast, sparsely populated, relatively unexplored continent existed when the Croton system was begun. By the time its last components were finished, the United States had become a major industrial power and most of our country's frontier was gone.

Two changes in public opinion and priorities began in the 1960s that would impact water supply systems across the country for the foreseeable future. The first was the rise of the environmental movement. Seeing the results of past abuses, many people began to challenge the entire "expand forever" philosophy. The second was an increased concern for water supply safety. This concern came to fruition with the passage, in 1974, of the Safe Drinking Water Act and its subsequent reauthorization in 1986. For the first time, the United States Congress mandated nationwide quality standards for drinking water and suppliers all over the country are working hard to implement them.

The Past: The Development of the System

New York City's present water supply system was developed over the last century and a half. Prior to that, the city relied on local wells and ponds for its water, but the rapid growth of its population in the first three decades of the 19th century exhausted the capacity of these local sources. Faced with this reality, the city began the development of its Croton system in the

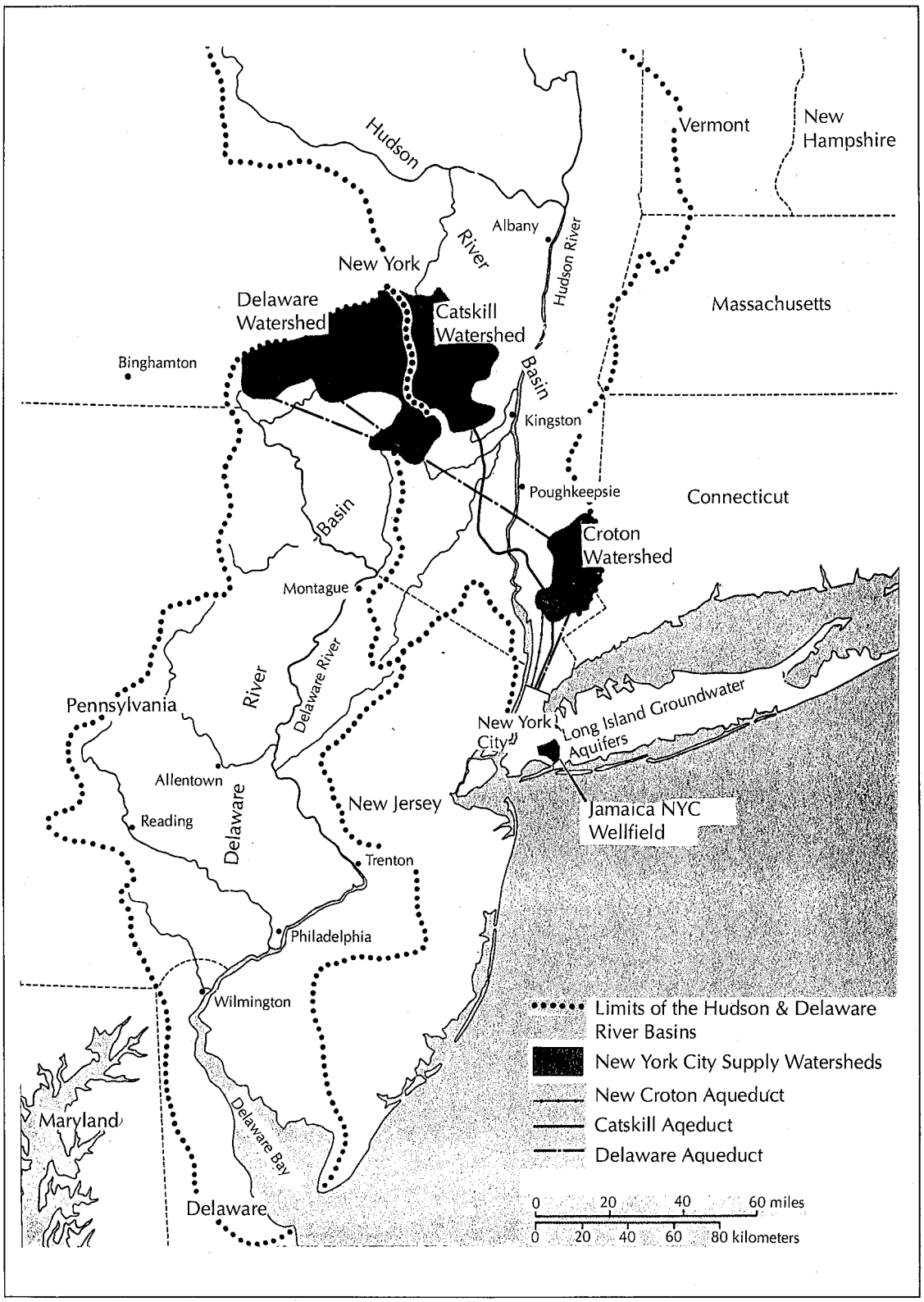

FIGURE 1. The New York City water use region.

1830s. In 1842, Croton water was first delivered to the city — a cause for wild municipal celebration. Yet, within a few short years, demand from a rapidly growing population exceeded reliable supply. This scenario was repeated over the next 150 years.

The main source of supply for the city today is surface water impounded in the three separate upland reservoir systems. The combined system includes 18 storage reservoirs and three controlled lakes having a total capacity of approximately 550 billion gallons. The separate collection systems were designed and built with a number of interconnections to provide operational flexibility and permit the exchange of water from one system to another. This capability mitigates the effects of localized droughts and permits full utilization of the total system capacity.

The amount of water that can be safely drawn from a watershed during the worst period in the drought of record is called the *safe (or dependable) yield*. It has been determined that the system could have furnished an average of 1,290 million gallons per day (mgd) during the drought of record in the mid-1960s. During periods of normal rainfall, watersheds supply more than the safe (or dependable) yield. Table 1 presents the safe yield and storage capacity for each of the three systems.

Water is conveyed to the city from the reservoirs in the Croton, Catskill and Delaware systems by gravity through large aqueducts and balancing reservoirs. Within the city, water is distributed through two major tunnels and four distribution facilities. A third tunnel is now under construction and will supplement the two city tunnels currently in use.

Approximately 97 percent of the total water supplied is delivered by gravity; only three percent has to be pumped in order to reach the highest areas of the city. As a result, operating costs of the present system are relatively insensitive to fluctuations in the cost of electrical energy.

Even though the system was developed by the city, it is truly regional in character since it supplies water to about 60 communities in Westchester, Putnam, Orange and Ulster counties that are located within the city's watershed area and along the routes of its aqueducts. These communities presently draw approximately 125 mgd, an amount that would be sufficient to supply the combined needs of the cities of Albany, Rochester and Syracuse.

TABLE 1
Water System Yield & Capacity

System	Safe Yield (mgd)	Storage Capacity* (billion gallons)
Croton	240	86.6
Catskill	470	140.5
Delaware	580	320.4
Total	1,290	547.5

*Capacity above minimum operating levels.

The Croton System

The Croton system consists of 12 reservoirs and three controlled lakes on the Croton River, its three branches, and on three other smaller tributaries (see Figure 2). Runoff from the Boyd's Corner and West Branch Reservoirs in the upper portion of the watershed is normally diverted to the Delaware Aqueduct. This diversion is possible because the Delaware Aqueduct passes right by the West Branch Reservoir and the hydraulic gradients at this point are equal. Water flows from the remaining upstream reservoirs through natural streams to downstream reservoirs, terminating at the New Croton Reservoir, the largest in the system. From the New Croton Reservoir, water is conveyed through the New Croton Aqueduct to Jerome Park Reservoir in the Bronx, and from there to the Central Park Reservoir in Manhattan.

Although the Croton watershed has an estimated safe yield of 240 mgd, only 140 mgd can be delivered by gravity and hydraulic pumping to the low areas of Manhattan and the Bronx. In normal years, therefore, approximately ten percent of the city's usage is supplied by the Croton system. However, by

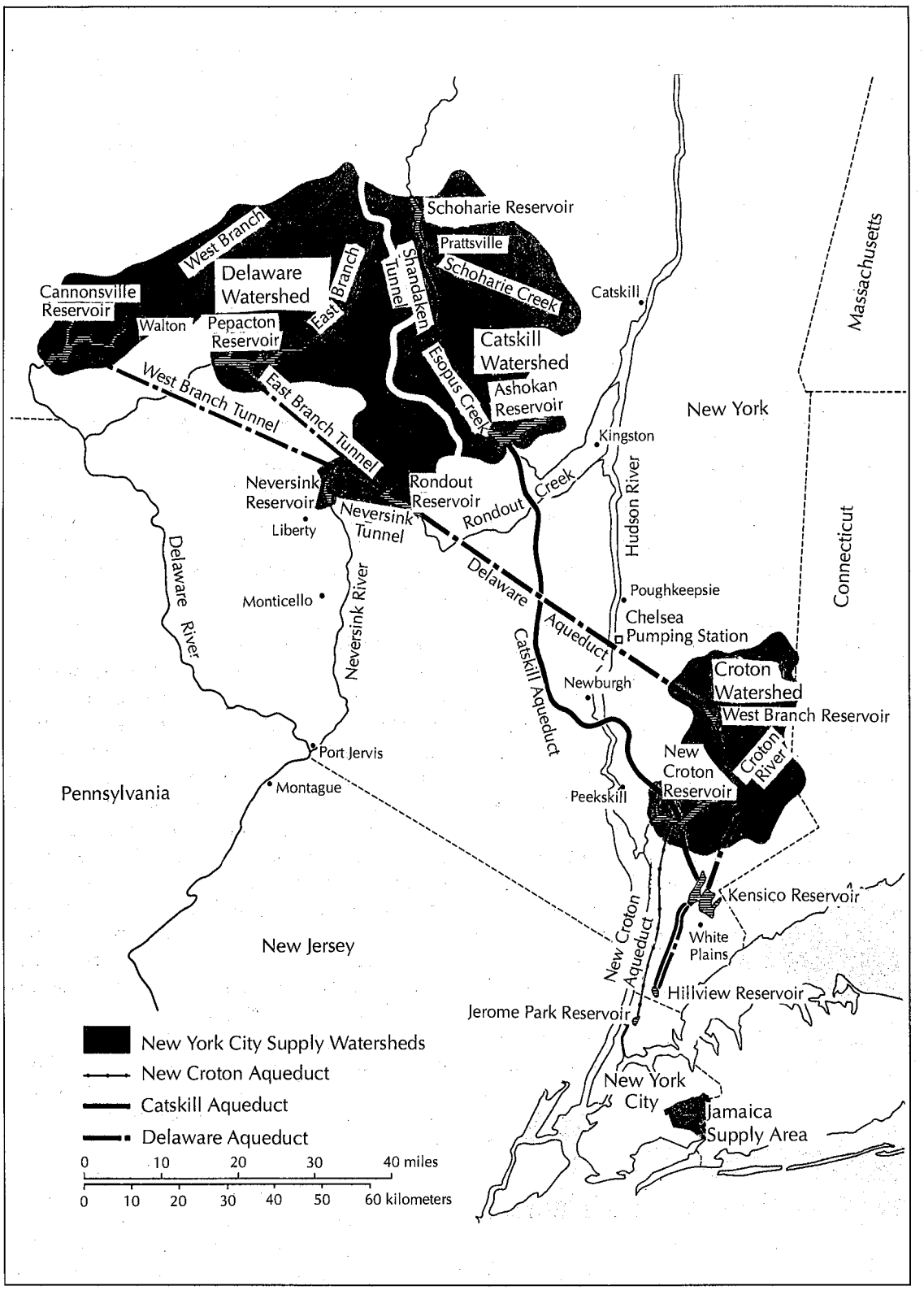

FIGURE 2. New York City's water supply system.

means of electric pumping and other distribution system manipulations, usage can be increased to about 280 to 300 mgd, approximately 25 percent of consumption, during periods of drought.

The Catskill System

The Catskill system was designed in the first decade of the 1900s, just as the Croton system was being completed. It was essentially finished in about eleven years, although a later reservoir, the Schoharie, was added in the 1920s. The scale and pace of the work dwarfed what had gone before it on the Croton system. Thousands of immigrants were recruited by contractors as they stepped ashore in this country, and were sent to work camps all along the route of the Catskill Aqueduct as well as to the construction sites of Olive Bridge and Kensico Dams. The presence of so many people in what were placid rural towns overwhelmed the resources of local law enforcement agencies and prompted the city to provide a separate police force in order to keep order. Looking back on that effort today, there is a tendency to be awe-struck by how so much was accomplished in such a short time.

All of these efforts were not universally approved. To develop the Catskill supply, the city had to displace hundreds of people whose families had, for generations, lived on land that would now be at the bottom of a reservoir. In doing so, the city often acted in a manner that was perceived as callous and high-handed by the local residents. A residue of ill will persists to this day and is one of the major obstacles that would have to be overcome if it were ever decided that additional sources were needed.

The Catskill watersheds occupy sparsely populated areas in the central and eastern portions of the Catskill Mountains and normally provide approximately 40 percent of the city's daily water supply (see Figure 2). Water in the Catskill system comes from the Esopus and Schoharie Creek watersheds, located approximately 100 miles north of lower Manhattan and 35 miles west of the Hudson River.

The Esopus Creek flows naturally into the Hudson River and drains an area of about 257 square miles. The Schoharie Creek drains into the Mohawk River from an area of 314 square miles. Most of the water from these two watershed areas is stored in the Ashokan Reservoir; the balance is stored in the Schoharie Reservoir.

The Ashokan Reservoir is formed by Olive Bridge Dam across the Esopus Creek. The Schoharie Reservoir is formed by the Gilboa Dam across Schoharie Creek at Gilboa, in Greene County, north of the Esopus Creek. The tributaries of the Schoharie Creek have their source at elevations almost 2,200 feet above sea level in the vicinities of Hunter, Windham, Prattsville and Grand Gorge in Greene, Delaware and Schoharie counties.

Water from Schoharie Reservoir is conveyed via the Esopus Creek and Shandaken Tunnel to Ashokan Reservoir where the Catskill Aqueduct begins. It is possible to divert water from the Catskill Aqueduct into the New Croton Reservoir to maximize the use of storage capacity.

The Delaware System

The Delaware is the newest, and largest, of the city's three water supply systems. It was developed over a period of 40 years, from the late 1920s to the late 1960s. Located approximately 100 miles north of lower Manhattan, it normally provides approximately 50 percent of the city's daily water supply (see Figure 2).

Three Delaware system reservoirs collect water from a sparsely populated region on the branches of the Delaware River: Cannonsville Reservoir (formed by Cannonsville Dam on the West Branch of the Delaware River); Pepacton Reservoir (formed by the Downsville Dam across the East Branch of the Delaware River); and Neversink Reservoir (formed by the Neversink Dam across the Neversink River, a tributary of the Delaware River). These reservoirs feed eastward through three separate rock tunnels — the West Delaware, East Delaware and Neversink — to Rondout Reservoir where the Delaware Aqueduct begins. Rondout Reservoir is not in the Delaware River watershed since Rondout Creek flows into the Hudson River.

Developing the Delaware system posed a new problem; one not encountered with the two earlier systems. In constructing, it the city would, for the first time, impound interstate waters. The city's efforts to use these waters resulted in a lawsuit brought by the downstream states of New Jersey, Pennsylvania and Delaware that challenged the city's plans. In 1931, the Supreme Court issued a decree apportioning the waters of the Delaware River. New York City was allowed to divert 440 mgd, but had to release sufficient water to maintain downstream flows in the Delaware River. The maximum amount the city had to release from Neversink Reservoir, the first stage of the Delaware system, regardless of the downstream flow targets, was 40 mgd.

When the Delaware system was expanded, a similar lawsuit resulted in an amended decree in 1954 that reapportioned Delaware River water. The system is operated under the terms of that decree to this day.

In 1961, another player entered the game — an Interstate-Federal Compact for the Delaware River Basin provided for the establishment of the Delaware River Basin Commission. Under the terms of this compact, the commission members (the states of New York, New Jersey, Pennsylvania and Delaware, as well as the United States Department of the Interior) could, by unanimous vote in an emergency, supersede the provisions of the 1954 Supreme Court decree.

The Effects of Drought

When the last stage of the Delaware system was designed, it was calculated that, on its completion, the total city system would provide a safe yield of 1,800 mgd. This calculation was based on the worst drought of record at the time. Unfortunately, before the system was finished in 1967, New York City and, indeed, the entire northeastern portion of the United States, suffered through a drought more severe than any previously experienced. Upon recalculation, the safe yield of the system was downgraded to 1,290 mgd — a 510 mgd reduction.

The magnitude of this reduction can be better appreciated when it is understood that it is larger than the safe yield of the entire Catskill supply (470 mgd). In 1965, at the height of the drought, it became obvious that if the city followed the terms of the 1954 decree rigidly, it would empty the Delaware reservoirs. This situation would have not only put the city at dire risk, but would also have ended all releases into the river downstream of the city's dams. The Delaware River Basin Commission responded to this situation in 1965 during the peak of the drought. The five parties of the commisision voted to supersede the provisions of the 1954 Supreme Court decree. It declared an emergency and modified the provisions of the decree. The amount the city was allowed to divert was reduced, but so was the amount it had to release downstream. In effect, the available supply was stretched out to get through the drought.

What happened during the drought left a lasting impression and a continuing dialogue was begun that continues to this day, all aimed at better managing and utilizing the water available. The decade of the 1980s, with its three droughts, led to further refinements of these efforts and it is fair to say that the system is far better prepared to weather a drought now than it was in the 1960s.

The Hudson River & The Chelsea Pump Station

Water may be pumped into the Delaware Aqueduct from the standby pump station at Chelsea, New York, which draws from the Hudson River. The Chelsea Pump Station has a capacity of 100 mgd. The second facility of its type to be situated at this location, the Chelsea Pump Station was reconstructed in 1965-66 under drought emergency circumstances and operated for approximately ten months during 1966 and 1967. It was then placed on standby status until 1981. In that year, again under drought conditions, the station was rehabilitated to full operating capacity, but the drought ended before it became necessary to place it into service.

In 1985, a recurrence of drought necessitated placing the station into service and it ran for approximately five months. In the spring of 1989, after a winter of extraordinarily low precipitation, the station was run for two weeks

and was shut down when heavy rains finally came.

The Present: How the System Operates

The city's water supply is transported through an extensive system of tunnels and aqueducts (see Figure 3). Croton system water is delivered from the New Croton Reservoir by the New Croton Aqueduct to the Jerome Park Reservoir in Manhattan. From Jerome Park and Central Park Reservoirs, and from direct connection to the New Croton Aqueduct, trunk mains carry water to the service area. The Catskill and Delaware Aqueducts convey water from Ashokan Reservoir and Rondout Reservoir to Kensico Reservoir and then to Hillview Reservoir in Yonkers. Kensico serves as a balancing reservoir; Hillview serves as a distribution reservoir. Water from the Catskill and Delaware systems is mixed in the Kensico Reservoir before it is conveyed to Hillview Reservoir from which water enters City Tunnels 1 and 2. Trunk mains carry water from tunnel shafts and from the distribution facilities (Jerome Park, Central Park and Ridgewood Reservoirs and the Silver Lake Tanks) to the various service areas.

Water Distribution

The water distribution system consists of a grid network of water mains ranging in size from six to 84 inches in diameter. It contains approximately 6,000 miles of pipe, 88,000 valves and 96,000 fire hydrants.

Slightly over half of the mains in the system are unlined cast iron, the primary construction material that was used before 1930. Between 1930 and 1970, cement-lined cast iron pipe was used, comprising about 40 percent of the water main mileage. Since 1970, the installed pipe material has been cement-lined ductile iron and now comprises about six percent of the water main mileage. The city has an extensive program for the replacement of water mains and spends about $100 million per year on this effort.

Water pressure is regulated within a range of 35 to 60 pounds per square inch (psi) at street level. Generally, 40 psi is sufficient to supply water to the top of a five- or six-story building. About 97 percent of total system consumption is normally delivered by gravity. It is necessary to pump only the remaining three percent to areas of higher elevation to keep the pressure within this desired range. High-rise buildings, of course, have their own internal pumping systems.

The distribution system in each borough is divided into three or more zones in accordance with pressure requirements. These zones are determined chiefly by local topography. The ground elevation in the city varies from a few feet above sea level, along the waterfront, to 403 feet above sea level at Todt Hill in Staten Island. The highest ground elevations in the other boroughs are: Manhattan, 267 feet; the Bronx, 284 feet; Brooklyn, 210 feet, and Queens, 266 feet. Various facilities provide storage to meet the hourly fluctuations in demand for water throughout the city, as well as any sudden increase in draft that might arise from fire or other emergencies.

Addressing Current Water Quality Concerns

The city's supply is known for the high quality of its water. Because of its inherent quality and the long periods of detention in its reservoirs, it has not yet been necessary to filter water from the system in order to reduce bacterial content and turbidity. The only treatment procedures routinely employed are detention, screening, the addition of caustic soda for pH control, chlorination for disinfection, and fluoridation. Additions of copper sulfate for algae control and alum for turbidity control are made occasionally, as needed.

Until recently, this level of treatment had proved to be more than sufficient to maintain water quality standards throughout the entire system. Population growth and commercial and industrial development within the Croton system have caused some deterioration of its water quality. The city is currently operating a demonstration treatment facility at Jerome Park Reservoir to develop design criteria for treating Croton system water. A full-scale treatment facility, the Croton Filter Plant, is scheduled for completion in the mid-1990s.

The system has five laboratories that moni-

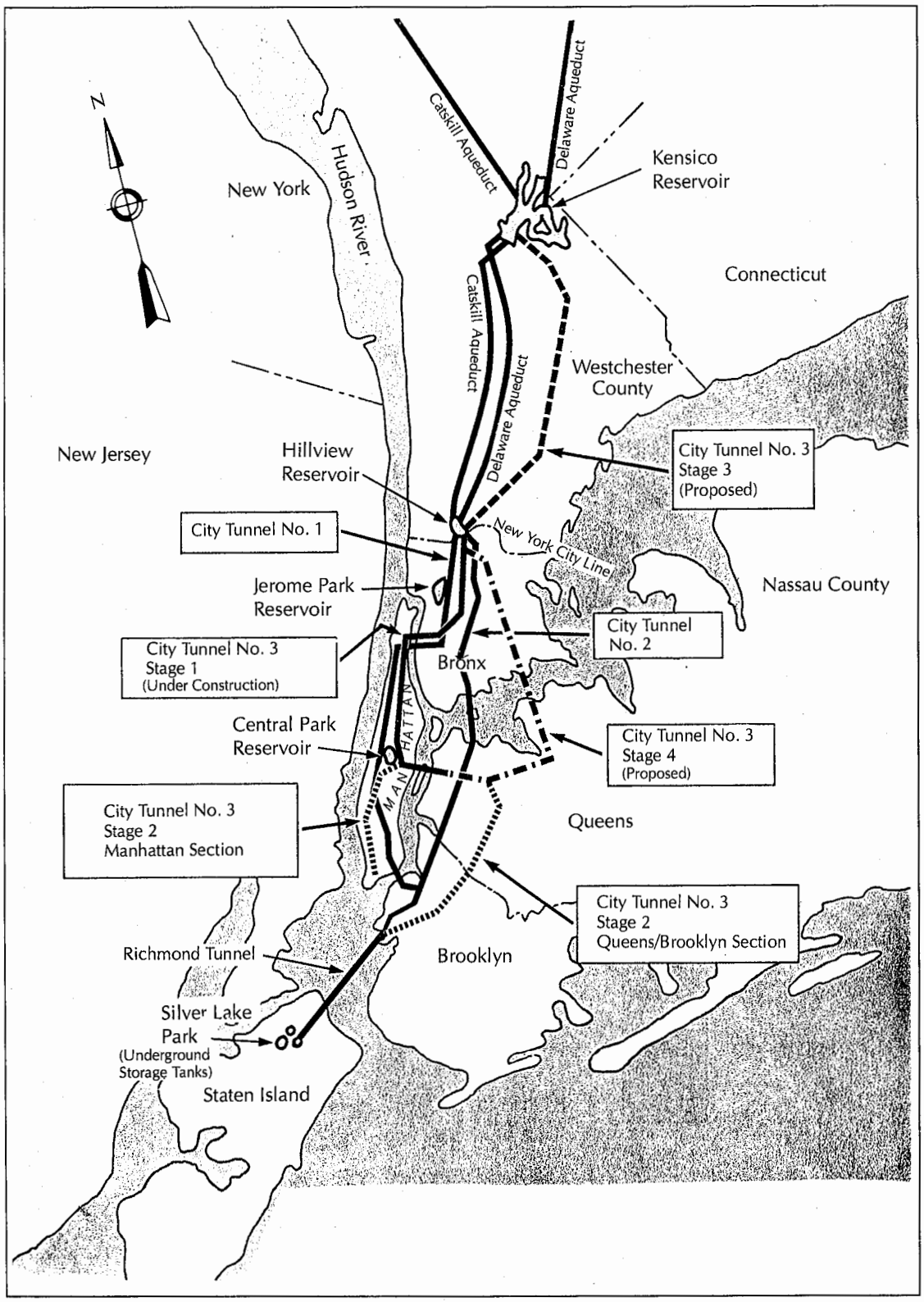

FIGURE 3. New York City's water supply transportation system.

tor water quality. They employ approximately 65 microbiologists, chemists and scientists. Over 40,000 samples per year are collected and 400,000 analyses are performed annually. Routine checks are made for 60 different substances, including heavy metals and trace organics. The monitoring program meets or exceeds federal and state requirements and has the capability to meet potentially more stringent requirements.

In addition to the monitoring program, watershed inspectors maintain constant surveillance of the watersheds. To further ensure high water quality, the system includes real estate adjacent to its reservoirs that has been acquired to prevent potential water contamination from sewage that would be produced if these areas were developed, and to control access to the reservoirs.

The Future: Where to Go From Here?

The frontier is gone. The potential for developing major new sources of supply is limited. The easiest supplies to develop have already been developed. New ones will be far more costly to bring on-line than those that exist now, both economically and environmentally. This reality leads to an inevitable conclusion: conservation must become the keystone in any new water supply construction program.

Concurrently, we must take much better care of what we have. America is a young country and we are only now seriously facing up to the hard task of reconstructing the great systems left to us by our predecessors. New York City is facing up to that task with an ambitious ten-year program that is estimated to cost approximately $8.5 billion (see Table 2). This sum, huge as it is, does not include the cost of developing any major new supplies or the cost of having to filter the Catskill and Delaware supplies. Two landmark actions were taken by the city to address these concerns.

Municipal Water Finance Authority

In 1984 the New York state legislature passed the New York City Water Finance Authority Act and a complementary law that created a public benefit corporation, the New York City Water Board. These laws were passed in order to achieve three main goals:

- To allow the city to sell revenue bonds to finance system improvements — something forbidden to cities by the New York State Constitution.
- To allow development of a modern accounting system that would clearly delineate the expenditures and revenues associated with the city's water and wastewater systems and that would set rates so that the revenues are sufficient to pay for the expenditures.
- To develop a long-range Capital Improvement Program, as shown in Table 2, to ensure the continued viability of the city's water supply and wastewater facilities. The amounts shown in Table 2 for New Source Development are for design and mid-term type projects, not for developing major new water sources.

Mayor's Intergovernmental Task Force

With New York City facing its second serious drought in five years, Mayor Koch, in July of 1985, formed an Intergovernmental Task Force to review the city's water supply system and make recommendations for its future needs. The task force was comprised of members from the United States Army Corps of Engineers; the United States Geological Survey; the New York State Departments of Health, Environmental Conservation, and State; the New York State Water Resources Planning Council; the various counties that surround the city; and, a number of city agencies.

The task force issued two interim reports. The first, "Increasing Supply, Controlling Demand," was released in February 1986. The second, "Managing for the Present, Planning for the Future," was released in December 1987.[1,2]

The report, "Increasing Supply, Controlling Demand," recommended:

"First, it is clearly necessary to study demand further and to refine projections for

TABLE 2
Capital Improvement Program

System Funds	(thousands) 1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	Total
Water Supply & Transmission											
Tunnel 3 Stage I	$ 11,091	$ 14,403	$ 5,060	$ 340	$ 0	$ 0	$ 0	$ 0	$ 0	$ 0	$ 30,894
Tunnel 3 Stage II	52,292	31,760	34,867	157,997	458,426	278,014	120,004	0	95,002	0	1,228,362
Tunnel 3 Stage III	0	0	0	0	10,468	0	0	0	0	0	1 0,468
Tunnel 1 Reconstruction	0	0	0	0	0	0	0	0	0	1,070	1,070
Misc. Expenditures	0	0	1,600	0	0	0	0	0	0	0	1,600
Subtotal	63,383	46,163	41,527	158,337	468,894	278,014	120,004	0	95,002	1,070	1,272,394
Water Distribution											
Croton Watershed Dam Safety Program	13,340	0	45,980	100,760	5,611	0	0	0	0	0	165,691
Croton Filter Project	3,306	9,999	0	225,939	0	27,680	0	22,467	0	0	289,391
Trunk & Distribution Main Extension	15,487	187,671	11,160	71,133	31,973	31,282	38,680	38,632	5,652	0	431,670
Trunk & Distribution Main Replacement	94,655	72,903	115,687	252,038	158,309	77,246	136,480	137,657	142,428	141,934	1,329,337
New Source Development	0	7,184	0	0	21,842	0	415,483	0	0	0	444,509
Upstate Improvements	2,822	4,570	6,367	17,160	21,627	6,617	3,480	17,168	3,727	3,856	87,394
Misc. Expenditures	7,335	1,207	0	0	0	0	2,714	0	0	0	11,256
Subtotal	136,945	283,534	179,194	667,030	239,362	142,825	596,837	215,924	151,807	145,790	2,759,248
Sewers											
Sewer System Extensions	70,555	45,377	44,271	60,252	57,962	51,070	50,000	53,977	46,958	30,470	510,892
Chronic Malfunction & Emergency Replacement	57,024	54,447	62,208	66,757	70,000	75,000	80,000	85,000	90,000	95,000	735,436
Programmatic Replacement	26,394	23,142	15,579	18,048	37,368	31,312	33,353	29,382	16,854	32,400	263,832
Replacement or Augmentation to Existing System	36,260	37,258	26,347	33,934	35,761	0	27,102	10,000	10,000	10,000	226,662
Program Response to Legal Mandates	7,813	29,351	16,405	6,667	11,750	18,104	0	2,000	0	1,000	93,090
Subtotal	198,046	189,575	164,810	185,658	212,841	175,486	190,455	180,359	163,812	168,870	1 ,829,912
Water Pollution Control											
Consent Decree Construction & Upgrading	41,046	2,205	42,102	135,530	119,440	112,000	112,000	0	0	0	564,323
Sludge Disposal	14,526	10,701	6,270	3,000	9,000	0	0	0	0	0	43,497
Plant Stabilization	1,650	1,100	1,749	1,799	33,499	22,100	21,800	0	0	0	83,697
Water Quality Mandates	24,423	10,740	23,000	318,100	19,500	281,301	246,000	69,999	10,000	0	1,003,063
Misc. Upgrading & Reconstruction	39,082	33,233	50,234	56,453	60,433	62,876	53,478	56,651	40,308	36,668	489,416
Subtotal	120,727	57,979	123,355	514,882	241,872	478,277	433,278	126,650	50,308	36,668	2,183,996
Equipment	13,387	9,482	22,867	7,209	4,780	5,019	5,268	5,531	5,807	6,097	85,447
Total System Funds	532,488	586,733	531,753	1,533,116	1,167,749	1,079,621	1,345,842	528,464	466,736	358,495	8,130,997
State & Federal Funds											
Water Pollution Control											
Consent Decree Construction & Upgrading	137,767	12,494	254,000	0	0	0	0	0	0	0	404,261
Total State & Federal Funds	137,767	12,494	254,000	0	0	0	0	0	0	0	404,261
	(thousands) 1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	Total
Total Funds All Sources	670,255	599,227	785,753	1,533,116	1,167,749	1,079,621	1,345,842	528,464	466,736	358,495	8,535,258

the city's future needs. An historical analysis of consumption as it relates to such factors as resident population, jobs, and the number of housing units should be developed. From this point forward, the city should monitor every aspect of demand on the water system not only to provide better data about future needs, but also to develop water-saving strategies.

"Second, the city should take every reasonable measure to control and contain water demand including metering and rate setting, leak detection and leak prevention, public awareness campaigns and school curricula, recycling and the use of groundwater for nonpotable purposes, and refitting buildings with lowflow, water-conserving fixtures. In the next decade every avenue of water conservation must be fully explored.

"Third, the city must acknowledge that, even with the best efforts at controlling demand, a supplemental supply of an additional 200 million to 300 mgd will be needed before the end of the century. The planning and development of this supply should proceed immediately and not await the results of the proposed demand study.

"Fourth, the city must prepare for the possibility that a much larger water development project, yielding 400 million to 1.2 billion gallons per day, will be needed in the long term. To lay the groundwork for such a large-scale development, the city should begin immediately to assess and select the most likely options, refining water yield, cost, time, and engineering estimates for projects such as the Hudson River High-flow Skimming Project, a reservoir system in the Upper Hudson Basin, or a system of canals and tunnels carrying water from the Great Lakes. The city should also do further study of the potential for recharging Long Island aquifers with reservoir water, and using the aquifer supply during droughts.

"To be successful, this comprehensive, long-term effort will require the dedication of additional resources to perform a consumption study, to develop water-saving strategies and conservation programs, and to evaluate potential water supply projects. The urgency and the momentum of the recent drought should not be lost as the reservoir levels return to normal. The city faces a very serious long-term water supply problem. If water demand continues on its current track, the city will be using 800 mgd more water in 2030 than it is using now. It must plan today to control this demand and to provide additional supply for it."

The report, "Managing for the Present, Planning for the Future," gave a detailed description of the actions taken in the period between the issuance of the two interim reports. In the section entitled, 'Summary and Next Steps,' it stated:

"This section provides a summary of the report, and describes the next steps that the Task Force believes should be taken in 1988 and subsequent planning periods.

"The Task Force organized its work during 1986-87 on the management and planning of New York City's regional water supply system through seven committees: Conservation, Demand, Groundwater, Hudson River, Long-Range Planning, Metering, and Water Quality and Watershed Management. Each of these committees focused on aspects of water demand and supply for the system. The work of the Demand, Metering, and Conservation Committees is described in Section 3, Demand Management and Conservation; and the work of the Water Quality and Watershed Management, Hudson River, Groundwater, and Long-Range Planning Committees is described in Section 4, Supply Potentials. The observations and recommendations presented are those that the committees submitted to the whole Task Force, and are in the form approved by the Task Force as its best response to Mayor Koch's charge to the group.

"The recommendations relating to Demand Management and Conservation focus on: monitoring the recently begun system demand study; the detailed implementation of the city's new metering program; and the implementation and improvement of the

range of conservation measures now in place and proposed. The recommendations relating to Supply Potentials focus on: improvements in the watershed management system, including water quality concerns; detailed investigation of additional withdrawals from the Hudson River in the general area of the Chelsea station, to prepare for the possibility that additional medium-term supply will be required; designs for more in-depth analyses of the possibility of using Brooklyn/Queens groundwater as a medium-term source; and, for more substantial long-term supply, should this be required, study of a Hudson River Basin project with or without flow augmentation.

"The Task Force has found that, because of the future effects of metering, conservation programs, demand growth in the city and among outside users, and the longer-term potential of changes in safe yield, the optimal package of management and development measures is not now known. However, the Task Force is convinced that the right course for the city is to pursue aggressively the management and planning measures recommended in this report. This course of action will enable the city, as the nature and extent of demands on the system become clear, to implement in a phased, efficient manner the appropriate range of management and development measures, including the continuation and strengthening of conservation measures now in place.

"The Task Force also observes that many of the management, planning, and development measures discussed in this report will take years to complete. For example, the new demand study is scheduled to last nearly six years; the implementation of metering in the city will take ten years; the detailed studies needed to decide on possible mid-term sources such as Hudson River withdrawals will take substantial periods of time. The completion of these and other tasks requires a strong commitment to effective planning over a long period both by the city and by intergovernmental forums such as the Task Force.

"In responding to the recommendations of the Task Force, insofar as supply is concerned, the city has prepared a request for proposal (RFP) whose scope includes the detailed studies required to determine the feasibility of two types of Hudson River projects: an expanded Chelsea-type facility and a larger facility having a capacity of 400 to 1,200 mgd. Eight proposers have survived the first screening and they will be submitting formal proposals by September 26, 1989.

"From the eight, three [in actuality four] finalists will be selected. These finalists will submit detailed proposals, including cost proposals, later this fall. It is anticipated that a contract will be signed with the successful proposer by the spring of 1990 and that work will start shortly thereafter."

It must be noted that current projections indicate that the contract will be signed this fall. Since the second interim report was issued, work has continued and the task force is expected to release its final report later in 1991. This report will give direction to the city's water supply program for the next several decades.

Addressing New Water Quality Control Standards

Concurrent with addressing a capital improvement plan and a demand management and supply augmentation study program, the city is also focusing on water quality issues and the possible need to filter its Catskill and Delaware supplies.

When the Safe Drinking Water Act was reauthorized in 1986, Congress directed the United States Environmental Protection Agency (EPA) to develop and promulgate a series of new regulations aimed at improving the quality of the nation's drinking water. Three of those rules seriously impact New York City:

- The Surface Water Treatment Rule (SWTR) issued in 1989.
- The Lead and Copper Rule issued in June 1991.

- The Disinfection By-Products Rule, scheduled to be released in 1992.

The SWTR sets criteria a supply must meet in order to avoid filtration. New York City believes it can meet these criteria. However, the State Department of Health, the prime agent in New York State with responsibility for implementing the provisions of the SWTR, has recently issued a draft proposal that would use SWTR criteria only for the next fifteen years and would mandate universal filtration by the year 2005. The city has opposed such a requirement, believing that it would drain money away from a recently approved water quality enhancement program aimed at protecting its watersheds. Should that opportunity be lost now, it will be gone for good. To the inevitable question of, "Why not do both?," there is a simple answer — money. It is estimated that a full scale plant, or plants, that would filter the Catskill and Delaware supplies will cost the city from $3 to $5 billion. Providing the resources to do this, while at the same time meeting its other obligations, will almost certainly preclude any large scale expenditures to acquire additional watershed lands or to purchase conservation easements for them.

The Lead and Copper Rule, while not as costly as the SWTR to implement, will bring with it large costs and problems of its own. In New York City's case, most of the corrosion control strategies proposed involve actions that could upset the delicate chemical and microbiological balance of the water in its distribution system. Caution must be the watchword here and the city is currently testing a number of corrosion-inhibiting compounds.

Another component of the Lead and Copper Rule will be a requirement to develop a lead service line replacement program if corrosion control does not reduce lead content to the levels required by the rule. It is estimated that it would cost approximately $500 million to replace every lead service line in the city, so effective corrosion control is vital.

The Disinfection By-Products Rule will, finally, determine the long-term viability of operating the New York City system as an unfiltered supply. In 1989, the United States EPA issued a draft proposal, the "Strawman Rule," to generate discussion. It is clear from reading this rule, and from discussions with staff and officials involved in the process of implementing it, that there will be at least a 50 percent reduction in the standard for total trihalomethanes (TTHMs) and that standards will be set for many compounds not currently regulated. Conformance to this rule would constrain the use of chlorine as a disinfectant.

Therefore, a situation is created where a SWTR primarily concerned with biological safety compels improving disinfection efficiency and increasing chlorine residuals; a Lead and Copper Rule compels increasing water pH to reduce corrosivity, a step that will have the two-fold effect of increasing TTHM production while at the same time reducing disinfection efficiency; and, a Disinfection By-Products Rule that will severely limit the use of chlorine as a primary disinfecting agent.

Conclusion

Will it be possible to walk the fine line between these constraints, furnish high quality water and remain an unfiltered supply? It is possible, and the effort may be well worth it. A situation exists where there is a last opportunity to retain a protected watershed, but time is running out. The next five to ten years will be crucial. If a well-protected watershed is acquired, it most probably will be preserved regardless of what additional treatment requirements may be imposed in the distant future. However, if a requirement for filtering by a fixed date is set now that opportunity will almost certainly be lost.

NOTE — *This article was originally presented as the John R. Freeman Lecture, sponsored by the Boston Society of Civil Engineers and the Ralph M. Parsons Laboratory at the Massachusetts Institute of Technology, on April 2, 1990.*

EDWARD C. SCHEADER *is Deputy Director of New York City's Department of Environmental Protection, Bureau of Water Supply. He is responsible for all planning, design and con-*

struction activities of the bureau, which has a ten-year capital plan totalling $4.5 billion. His current major project is the Croton Filter Plant, New York City's first water filtration facility. He is a life member of the American Water Works Association, as well as a member of the Association of Dam Safety Officials, the American Water Resources Association and the Association of Metropolitan Water Agencies.

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A Model Coastal Zone Building Code for Massachusetts

Residential and small commercial structures must be designed to withstand static flood water level forces, hydrodynamic velocity forces and high winds in coastal areas.

AD HOC COMMITTEE ON COASTAL ZONE BUILDING CODES OF THE BSCES WATERWAY, PORT, COASTAL & OCEAN ENGINEERING TECHNICAL GROUP

In June 1989 the Boston Society of Civil Engineers Section/ASCE (BSCES) Waterway, Port, Coastal and Ocean Engineering (WPCOE) Technical Group executive committee voted to form an ad hoc advisory committee on coastal zone building codes. The purpose of this ad hoc committee was to review the adequacy of the existing provisions of Section 2102.0 of the Massachusetts State Building Code (MSBC) on construction requirements in flood plains and coastal high hazard areas, and to make recommendations for its improvement in the light of recent coastal storm experience and coastal engineering developments. It was determined at an early stage of the committee's work that the development of an independent Model Coastal Zone Building Code for Massachusetts would be informative and could be subjected to general peer review prior to making definitive recommendations for incorporation into the MSBC. The model code was presented in final working draft form at the Coastal Zone Construction Seminar sponsored by the WPCOE Technical Group on April 6, 1991, and is commented on below.

Introduction

Building structures that are located in coastal flood plains and coastal high hazard areas should be designed to resist (or preferably to avoid) the forces imposed by static flood water levels and, in high hazard areas, by hydrodynamic velocity forces. In addition, these structures are typically exposed to unusually high winds that are unimpeded by flat terrain and overwater exposures. Their location along the ever-changing coastline makes them vulnerable to both short and long term erosion related damage. Recent coastal storm experience demonstrates that residential structures in particular are either inadequately designed and constructed, or they are improperly situated to cope with flood water forces and high wind exposures. For example, in the "Blizzard of 1978" over 2,000 residential homes were de-

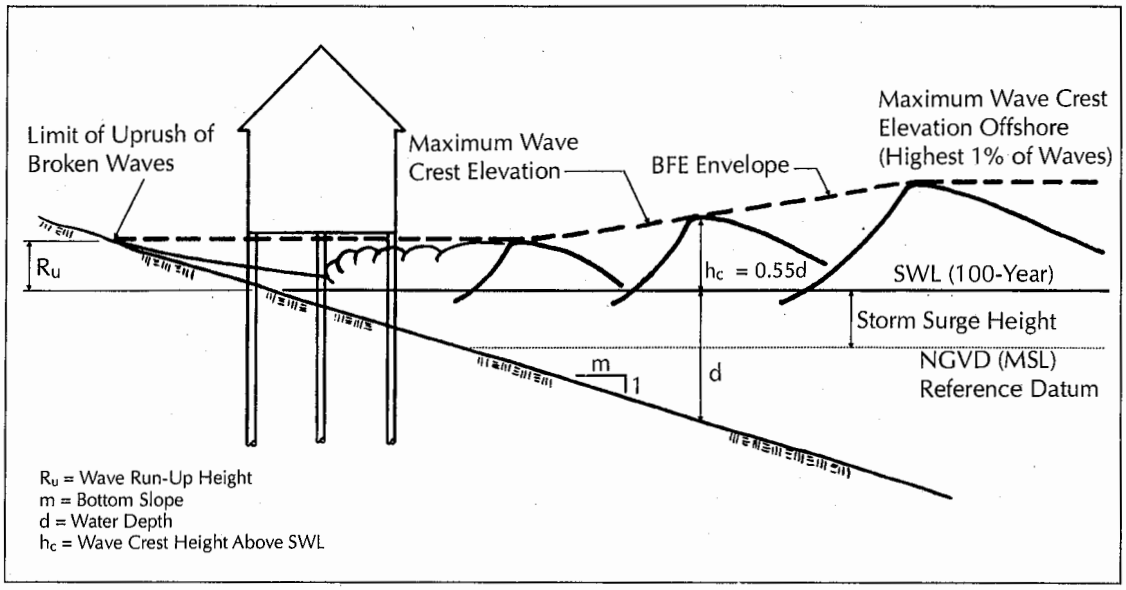

FIGURE 1. Base flood elevation definition sketch (FEMA criteria).

stroyed, over 10,000 damaged and up to 30,000 were otherwise affected by the storm's fury.[1,2] Structural investigations along the South Carolina coast,[3,4] in the wake of Hurricane Hugo in 1989 have demonstrated the same lessons learned from past storm investigations. Damage on the United States mainland from that hurricane was estimated to exceed $7 billion in losses and 15,000 homes were destroyed.

The committee reviewed these and other findings and various other attempts at producing local coastal zone codes[5-10] and adapted applicable requirements to Massachusetts. As of 1988, Massachusetts insured coastal property exposures totaled approximately $180 million. The incremental cost of constructing one- and two-family dwellings in order to meet prescriptive coastal wind requirements is on the order of 1.5 to 4 percent of the cost of normal construction.[11] Construction of an elevated pile foundation designed to meet coastal high hazard area requirements are estimated to result in an increased cost on the order of 10 to 20 percent over that of normal at-grade construction, under most circumstances.

Massachusetts is nearly unique in having two distinctive types of coast: *i.e.*, the coast of New England north of Cape Cod has many rocky headlands and steep cobble beaches typical of Atlantic Boreal regions, while also possessing notable barrier beach systems; south of Cape Cod the coastline generally consists of long sandy beaches typical of the Mid-Atlantic regions. The Massachusetts coast north of Cape Cod is particularly vulnerable to extra-tropical winter storms known as "Northeasters,"[12,13] while the southern Massachusetts coast is more likely to be severely affected by tropical storms known as "Hurricanes."[14,15] Since the meteorological characteristics of such storms may have a profound influence on site specific conditions to be encountered (*e.g.*, the simultaneous occurrence of maximum wind speeds from a given direction and maximum storm surge), flood levels and wave heights should be taken into consideration when siting a structure and determining design criteria. Recent projections of accelerating rates of sea level rise[16] that have not historically been considered by other codes have been incorporated into the model code.

The Federal Emergency Management Agency (FEMA) has produced Flood Insurance Rate Maps (FIRMs) along with Flood Insurance Study Reports for most Massachusetts coastal communities.[17] The FIRMs specify minimum base flood elevations (BFEs) associated with a 100-year return period or an event having a one percent probability of occurrence in any given year. The BFE represents the maximum still water level (SWL) associated with storm surge

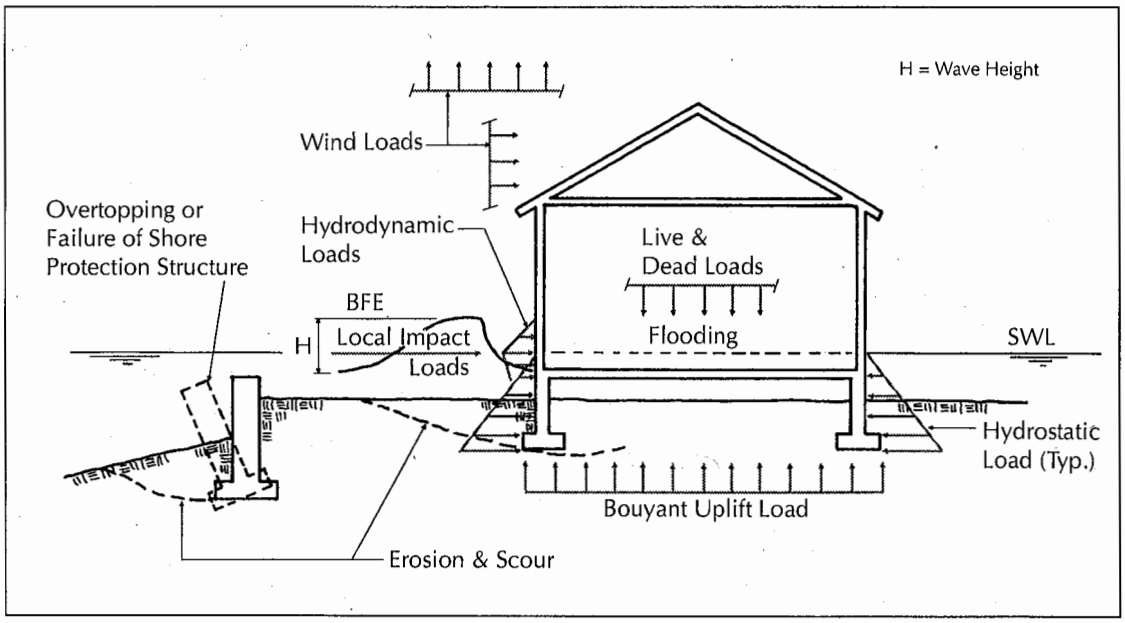

FIGURE 2. Summary of storm related loads acting on a coastal building.

tide elevations for "coastal flood plains" and the maximum wave crest or run-up elevation for "coastal high hazard zones" designated as V-Zones on the FIRMs (see Figure 1). The FIRM BFEs are recognized by the current MSBC, however the model code has attempted to clarify their application with specific FEMA design guidance.[18] Additional FEMA guidelines and general design guidance can be found in a number of sources.[19-28]

Coastal Storm Damage

Figure 2 illustrates the types of forces acting on a building under coastal storm conditions and Table 1 summarizes the typical structural problems in approximate ascending order of severity and descending order of frequency. The primary destructive agent is moving flood waters that result in buoyant uplift, impact and velocity forces (including impact of waterborne debris, "flotsam"), and erosion and scour of foundation materials. The deleterious effects of the latter cannot be overemphasized.

Figure 3 illustrates the damage to a coastal residence due to the infamous Blizzard of 1978. Water velocity forces are best dealt with by designing to avoid them rather than to resist them. The solution is relatively simple in principle: *i.e.*, raise the enclosed building structure above the highest expected flood level including the wave crest height and/or run-up height above the SWL or BFE. Doing so often results in buildings constructed on elevated pile or pier foundations (see Figure 4) that allow flood waters to move essentially unobstructed below the first habitable floor elevation. Below the first floor, "break-away walls" may be provided for aesthetic reasons or to provide partial enclosure for storage or parking areas.

It is essential that adequate depth of foundation piles or pier footings be provided in order to allow for both the effects of long term shoreline erosion and the scour depths that may occur during a major storm event. Both the foundation and overall structure must be adequately anchored and braced to resist lateral and uplift forces due to combined wind and water forces. Appurtenant structures — such as stairs, porches, car ports, *etc.* — are often a source of structural problems since they are often lightly constructed and poorly attached to the main structure. Their failure may cause damage to the main building structure by causing damage or weakness at points of connection and/or by causing impact damage as flotsam.

A few basic design principles for sound coastal zone building construction can be summarized:

TABLE 1
Typical Coastal Zone Building Structural Problems

Severity	Frequency	Type of Damage	Primary Mechanism	Structural Deficiency
Most Severe	Least Frequent	Foundation Failures	Erosion/Scour Buoyancy	Inadequate Foundation Depth/Scour Allowance, Inadequate Uplift Anchorage
		Roofing & Cladding Failures	Wind	Inadequate Connections for Local Suction Pressures
		Localized Failures (Especially Appurtenant Structures)	Wave & Current, Impact of Waterborne Debris	Exposure to Wave & Water Impact. Inadequate Construction & Connection of Appurtenant Structures
Least Severe	Most Frequent	Flooding (Water Damage)	Static Water Level Rise	Inadequate Elevation Above Storm Surge & Wave Crest Elevation

- Elevate the lowest floor above the maximum storm surge elevation plus wave crest/run-up elevation and set the structure back from breaker zone as far as possible.
- Provide deep foundation with adequate scour allowance, lateral capacity and uplift resistance.
- Provide adequate bracing in order to transmit lateral loads throughout the structure with particular attention to connection details.
- Provide adequate wind anchors for roofing and cladding in order to resist localized suction pressures.
- Eliminate and/or reduce the number of projections and appurtenant structures. Provide suitable connections where appurtenant structures are unavoidable, use storm shutters on exposed windows, *etc.*

Although the model code applies in general only to habitable residential and commercial building structures, a few comments on the application of shore protection structures — *i.e.*, seawalls, bulkheads, revetments, *etc.* — are warranted. Unfortunately, many such structures are poorly constructed and inadequately designed, and may lead to a false sense of security and/or may actually cause the kinds of problems that they were designed to solve (see Figure 3).

The investigation of structural damage has lead to the following conclusions:

- Buildings should not be founded directly on seawalls or revetments.
- Seawalls, bulkheads and revetments, *etc.*, should not be counted on to protect buildings unless they are properly engineered and constructed by specialists.
- Improperly designed or constructed shore protection may pose a hazard to buildings. For example, ballistic stones due to under-designed mound structures, walls that accelerate erosion or reflect wave energy, *etc.*

FIGURE 3. Wave impact damage to timber frame structure at Gray's Beach, Manchester, Massachusetts, due to the "Blizzard of 1978." Note the total destruction of the concrete seawall.

Commentary on the Proposed Model Code

The model code consists of seven sections (see Table 2 for a listing of the code's contents), the composition and intent of which are briefly described as follows:

Section 1 provides the general background, scope and purpose of code, as well as definitions of key words and acronyms. It also defines designated areas and projects as well as compliance with other regulations and standards. Since the model code is essentially a "performance" type code, Section 1 lists references that provide specific design guidance and that are considered an integral part of the code.

Section 2 describes siting requirements. Although the code is not intended to perform a regulatory function, it recognizes certain geographic and topographic regions

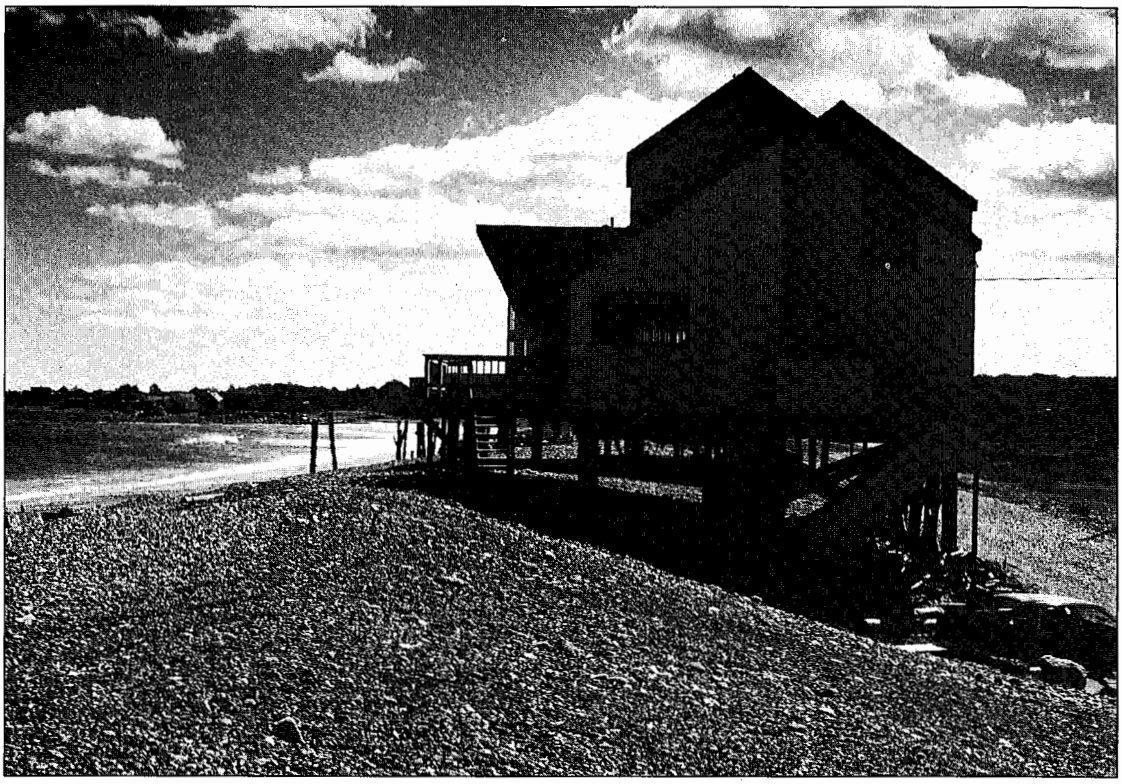

FIGURE 4. House on an elevated pile foundation at Mann Hill Beach, Scituate, Massachusetts. A previous home on this site was destroyed by the "Blizzard of 1978."

where local geologic conditions — *i.e.*, long term shoreline change, short term erosion potential and/or unsuitable foundation materials — would render any construction vulnerable to significant damage or destruction over its life regardless of the stringency of structural design requirements. Such areas, therefore, are prohibited for new building construction on the basis of technical design considerations. The application of shore protection structures in relation to building construction is also described. The model code further recognizes a distinction between coastal "flood plains" that are exposed primarily to static flood water levels (FEMA designated A-Zones) versus coastal "high hazard areas" that are additionally exposed to wave action and higher velocity flood waters (FEMA designated V-Zones).

Section 3 outlines criteria and provides general design guidance for the evaluation of structural loadings. This section recognizes the site specific nature of storm loadings with particular regard to the probability of simultaneous occurrence of maximum water levels, with winds and waves from a given direction, and to the meteorology of storm conditions. Figure 5 is reproduced from the model code and defines three major storm zones. The design windspeeds shown represent the 100-year "fastest mile" hurricane windspeeds from any direction after Batts, *et al.*, for coastal exposure.[29] Structures should be designed to resist the forces associated with the given windspeed from any direction in the given zone. The zones are dictated by the nature of storm driven waves and flood levels that are most likely to occur. Zone I is primarily subject to the highest storm tide elevations and protracted wave action caused by Northeasters, whereas Zone III is primarily affected by Hurricanes and Zone II is vulnerable to both types of storm. Section 3 also requires that the recently predicted accelerated rate of sea level rise[16] due to global warming should also be

TABLE 2
Model Coastal Zone Building Code for Massachusetts

Contents

considered in evaluating the maximum SWL to be applied to the structure's final design. This section further prescribes certain minimum loadings, such as for impact and velocity forces on cross bracing, where the selection of appropriate design criteria is otherwise vague. The U.S. Army Corps of Engineers' *Shore Protection Manual* is cited in this section for the evaluation of hydrodynamic loads[30] and ASCE 7-88 for the evalu-

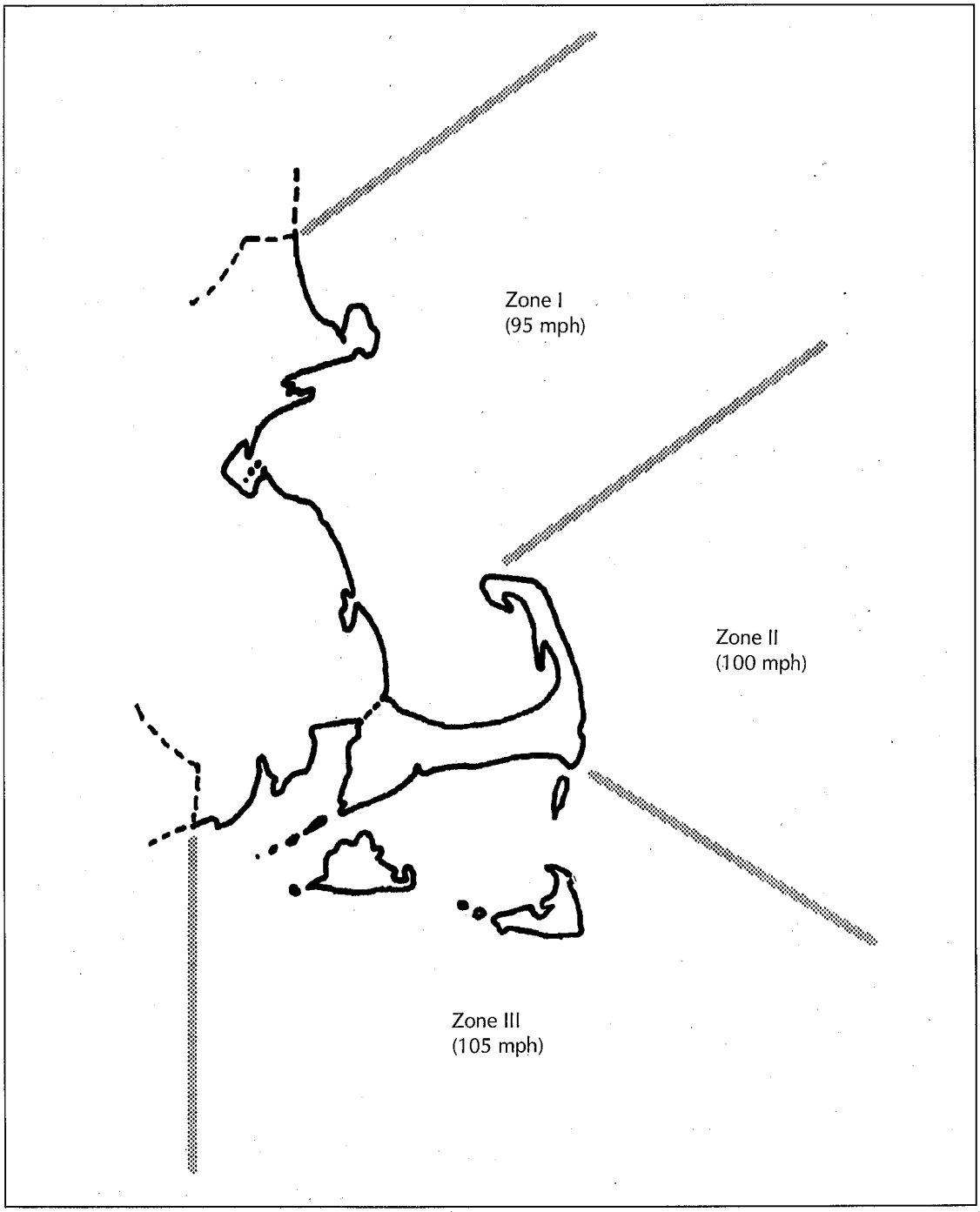

FIGURE 5. Coastal Massachusetts storm hazard zones (basic design windspeeds are shown in parentheses).

ation of wind, seismic and all other structural loadings.[31]

Section 4 provides specific structural design requirements in an essentially "performance" type format. Certain minimum dimension "prescriptive" requirements are cited, primarily for single family and duplex residential construction where detailed en-

gineering investigations would be too costly.[18] The requirements of this section emphasize proper connection strength and details, lateral bracing systems and uplift anchorage. Allowable stresses under given load combinations are addressed. Recommended design load criteria for break-away walls are promulgated in order to assure that such walls do not impose excessive loads on the main building structure prior to their failure. The FEMA *Coastal Construction Manual* is cited in the section for further structural design guidance.[18]

Section 5 provides specific foundation design requirements again in an essentially performance type format. This section, however, is prescriptive in prohibiting the use of slab on grade and shallow wall type footings in coastal high hazard areas. The use of fill for structural support is not allowed and minimum dimensions are prescribed for piles and piers in coastal high hazard areas. Minimum pile embedments and scour allowances are also considered.

Section 6 provides additional design requirements for the location and security of utility systems and for mobile homes.

Section 7 requires certification of compliance with the general code requirements including as built records of floor elevations, pile penetrations or footing depths, and receipt of design plans and specifications stamped by a qualified engineer or architect.

Conclusion & Implementation

There is a need for a coastal zone building code for Massachusetts and the purpose of the proposed model code should fulfill that need. The final model code can either be attached as an appendix to the MSBC or its salient features could be rewritten into the existing MSBC format of Section 2102.0. Another option would be for the model code to serve as an attachment to Article 34 of the MSBC for One and Two Family Dwellings. In the case of one and two family dwellings, the model code could be rewritten in a more "prescriptive" format in order to reduce engineering design costs, review time and effort by building officials while assuring substantial compliance with the performance requirements of the model code. Whatever the final disposition of the model code, the working draft and this discussion of its development should contribute to a better understanding of the peculiar design requirements of buildings that are situated along the coast and that are subject to coastal flood hazard.

NOTE — *A working draft of the proposed model code was presented at a BSCES/WPCOE Group Coastal Zone Construction Seminar on April 6, 1991, and a copy is available for review at the BSCES offices at the Engineering Center, 1 Walnut St., Boston, MA 02108. Review and comment on the code is welcomed and any comments that are received will be considered when the model code is finalized.*

ACKNOWLEDGMENTS — *The Ad Hoc Committee on Coastal Zone Building Codes of the BSCES Waterway, Port, Coastal and Ocean Engineering Technical Group executive committee consists of seven members who together represent over 100 years of experience in waterfront and coastal construction. The members are listed with technical expertise from several key disciplines noted: Richard Baker, Physical Oceanography, Metcalf & Eddy, Inc.; John Bannon, Structural Engineering, Childs Engineering Corp.; Louis Nucci, Geotechnical Engineering, Nucci Vine Associates; David Porter, Structural Engineering, Childs Engineering Corp.; Lester B. Smith, Jr., Coastal Geology, Daylor Consulting Group, Inc.; Peter J. Williams, Coastal Engineering, Gale Associates, Inc.; John W. Gaythwaite, Chairman, Structural Engineering, Maritime Engineering Consultants.*

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Lessons from Hurricane Hugo: The Need for Codes & Performance Criteria in Marinas & Coastal Structures

Formulating much-needed codes & standards for marina & coastal structure design requires the cooperative efforts of public agencies, engineers, building officials, legislators & insurers.

JON GUERRY TAYLOR

Since Hurricane Hazel struck in 1954, South Carolina had been spared a direct hit by a major hurricane until September 22, 1989, when Hurricane Hugo slammed into the South Carolina coast. The eye of the storm advanced through Charleston Harbor, passing inland through the middle of the state (Sumter) towards Charlotte, North Carolina (see the map presented in Figure 1). Hugo was a one in 50-year event in Charleston, but was a one in 250-year event in Sumter and in Charlotte. Hurricane Hugo jumped from a Category II hurricane (winds of 96 to 110 mph) to a Category IV hurricane (winds of 131 to 155 mph) just prior to making landfall. The tidal surge accompanying the hurricane varied from 12 feet in Charleston Harbor at the center of the eye of the storm to approximately 20 feet on the north advancing edge of the eye of the storm (Awendaw, Cape Romain and McClellanville). From McClellanville, the tidal surge averaged about 12 feet to the Little River Inlet at the North Carolina/South Carolina boundary. South Carolina had less than 24 hours of lead time for hurricane preparation and the evacuation of barrier islands such as Isle of Palms, Sullivans Island, Kiawah and Folly Beach.

Marina facilities in South Carolina vary from small (from 30 to 50 slips) to medium (from 100 to 200 slips). Because of the normal tidal range (from eight feet in Hilton Head to five feet in Charleston), many marina facilities utilized floating docks constructed of timber, concrete or aluminum. Private and commercial marinas

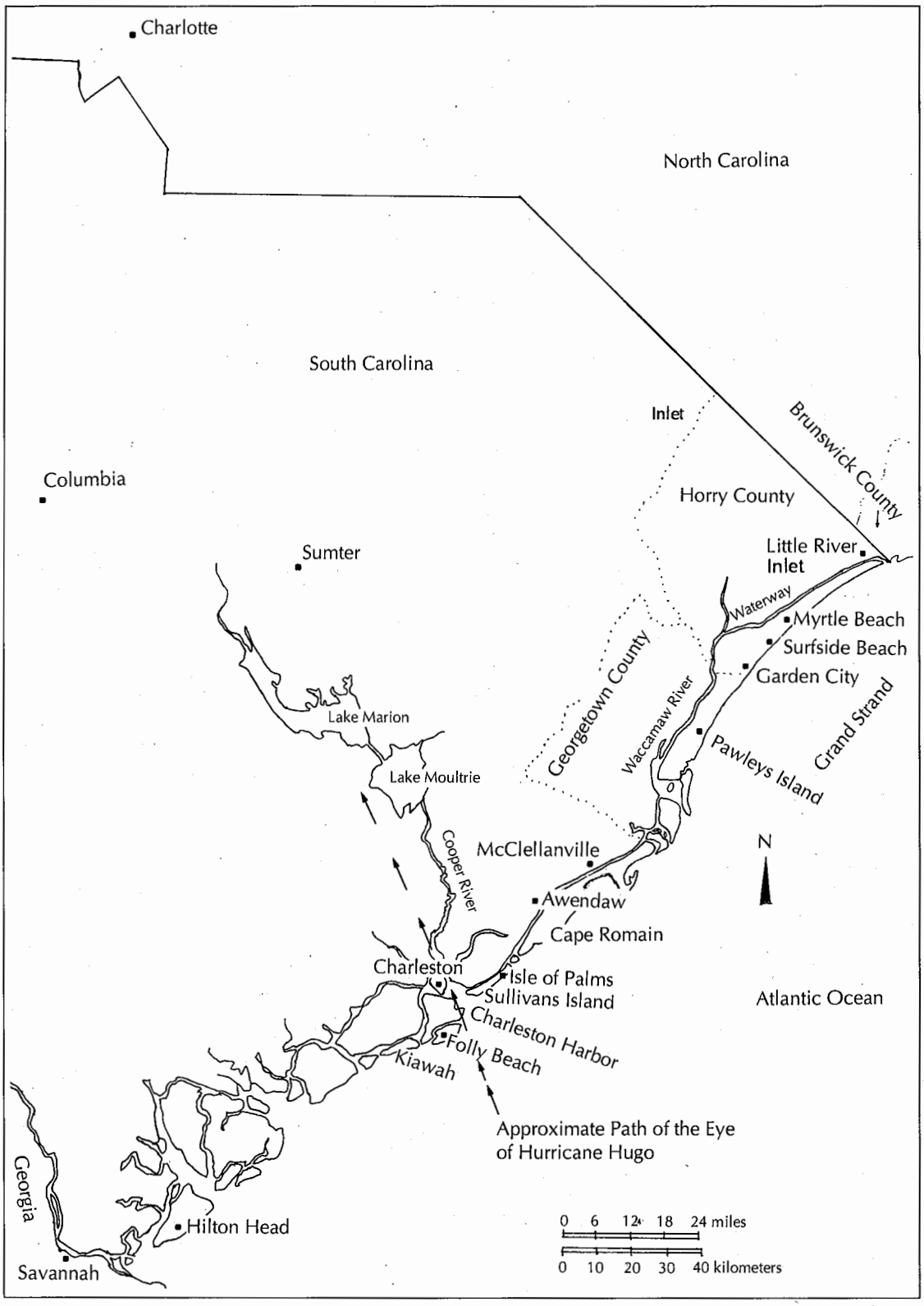

FIGURE 1. The approximate path of the eye of Hurricane Hugo through South Carolina.

were relatively new in South Carolina. Many of these marinas had been constructed in the past ten to fifteen years and were often operated by personnel with little or no experience with hurricanes. Public and private fishing piers were a long established usage in the Grand Strand area (Garden City, Surfside Beach, Myrtle Beach, *etc.)* where tourism is a major economic factor.

At that time South Carolina had no design standards for private residential piers, public or private fishing piers, or marinas. None of the coastal counties in the state had coastal structural or marina design standards or codes. However, the South Carolina Coastal Council had a written hurricane evacuation requirement in its *Operations and Maintenance Manual.*[1] Conformance to this document is a requirement for obtaining a marina permit in the South Carolina Coastal Critical Zone. However, no other public agency required plans for hurricane preparation, design, evacuation or recovery. None of these agencies required the participation of registered design professionals in the planning and design of marina facilities.

Immediately after Hurricane Hugo struck South Carolina, a comprehensive investigation and analysis of the performance of marina structures and operations during the hurricane was undertaken.[2] The hurricane/marina experience was further broadened by a visit to Puerto Rico in order to investigate other hurricane damaged marinas. In addition, other damaged structures and their reconstruction after the storm were monitored. These other structures included retaining walls, rip rap erosion control, fishing piers and residential floating dock and pier construction. Some of the more prominent design and construction failures are discussed here, with recommendations for minimum design considerations that could become the basis for the further development of minimum codes for marina structures in hurricane-prone areas.

Piles

The local practice in pile design in South Carolina favored the use of timber piles in marinas and coastal structures such as fishing piers, residential fixed piers and floating docks. Many of these piles were not designed by engineers since they were only one feature of contractor designed/installed structures. Soil conditions vary from sandy soils (Unified Soil Classification SM-SP) to organic clay (Unified Soil Classification OL, locally referred to as "pluff mud") over deeper marl (brownish green calcareous clay with slight sand content — Unified Soil Classification, MH). Residential pier structures — some as long as 800 feet — usually utilized jetted piles in lieu of driven piles. Lateral loads from wind and waves were often not even considered when determining pile sizes and pile penetration. The use of ten-inch and 12-inch square prestressed piles for horizontal loads had been growing in recent years. A locally accepted top of pile elevation was 12.0 feet above mean sea level (MSL).[3] This elevation would accommodate only a six- to eight-foot tidal surge (assuming the storm hits at high tide — adding a 1.5-foot freeboard for floating docks and a two-foot wave action during the storm).

By far, the most frequent failure in coastal and marine structures in South Carolina during Hurricane Hugo was pile failure. Both independent piles and piles within systems experienced a disproportionate number of failures. Many of the piles in fixed-pier systems failed from "pullout" due to uplift forces from wind and tidal surge. Others either broke at or near the mudline or leaned over from horizontal force. A final failure of piles in floating dock systems was found when piles did not have adequate height to restrain the floating docks during the tidal surge (see Figure 2).

Recommendations. A minimum timber pile size (tip) in coastal structures should be 25 inches in circumference for timber piles, and 12-inch square for prestressed concrete piles. Minimum pile penetration requirements based on local soil conditions should be developed. Pile penetrations of 20 feet for clay and 15 feet for sand should be the absolute minimum, regardless of local soil conditions. The minimum height of piles for floating dock structures should be set to withstand a designated hurricane's wind intensity and storm surge — *e.g.*, the minimum height for piles experiencing a 50-year storm occurring at high tide would be the mean high water elevation plus the height of the 50-year storm surge plus 1.5 feet for dock free board and two feet for wave action. Driven

FIGURE 2. Hugo's storm surge took floating docks over the top of anchor piles and washed them ashore.

piles should be preferred over jetted piles and the jetting of piles in marinas and commercial pier construction should be permitted only under the supervision of a professional engineer. The use of old telephone poles for piles in salt and brackish waters should be prohibited. Piles for marina structures and commercial pier structures should be designed by a professional engineer.

Fixed Piers (Residential)

The coast of South Carolina includes large areas of tidal marsh that must be traversed with fixed piers before reaching deeper water suitable for navigation. Local dock builders and residential marine contractors utilized light structural components (nails, 0.375-inch galvanized bolts, butt splices on the nailers and side members, and toe nailed connections instead of straps). These structures served adequately while under vertical loads only. However, when confronted by the high winds, wave action and tidal surge caused by Hugo, they failed. Where pile penetration and pile size was adequate to withstand the uplift forces, the decking and handrails on the fixed piers were the first to go (see Figure 3). In some cases, if the pile cap connection was not adequate, the total deck structure (including stringers) with the handrail intact lifted off the pile or pile cap. Often, where pile penetration and size were not adequate to withstand the uplift forces, the entire structure, including piles, rolled over. This type of failure was especially prevalent in residential fixed piers that utilized small jetted piles in marsh areas (pluff mud soil condition).

Recommendations. There are many individual site conditions that have to be addressed on numerous residential installations. Because of their small scope, these projects usually do not have the involvement of a professional engineer. Often, a local building permit is not even required, so residential pier and dock projects escape local inspection by code enforcement officers. There should be some minimal standards for fixed-pier design. The only entities that are involved with all of these projects are the environmental permitting agencies (U.S. Army Corps of Engineers, Coastal Zone Man-

agement, *etc.)*. These agencies could promulgate suggested minimum guidelines of construction for these facilities.

Floating Docks

When anchored adequately, floating docks can move, deflect and relieve stresses. As previously mentioned, some floating docks floated over the tops of piles during Hugo's storm surge. Many floating docks failed from the loads transmitted by the boats that were tied to them. In addition, if the boats were not properly secured, they rubbed the docks, tore away from their moorings and caused impact damage throughout the marinas. Some floating docks failed as a result of impact from heavy floating debris (one floating tire breakwater broke away from its anchor and caused substantial damage in the marina it was supposed to protect). All floating docks suffered damage during Hurricane Hugo. However, the commercial "factory produced" docks fared better than locally produced docks and they were easier to repair or replace after the storm. Connections at the finger/main walkway junction exhibited weakness. Properly designed knee braces helped to sustain the lateral and twisting loads from boats anchored in the berths. Even concrete docks sustained damage through the wale system when they heaved against piles. Other dock systems tore apart at the connections between units (modules). From field observations, it appears that most floating docks failed after coming loose from their moorings (piles). Once they were taken out of their alignment, they were torn apart by twisting and bending. In many cases, the flotation billets were torn away from the structure (see Figure 4). Utilities in the dock systems were severed. Small cleats (eight inches and less) broke or were ripped from their moorings. Outside pile guides were torn from their moorings or were twisted when the docks broke loose.

Recommendations. The attention to adequate mooring (piles) and mooring devices (pile guides) will contribute substantially to the performance of the dock system. The piles must be high enough to restrain the floating docks during the storm surge. The mooring attachments to the docks (pile guides) must allow free movement while keeping the docks in alignment. This combination of design considerations will keep the system intact throughout the storm cycle. The stresses caused by boats beating against the floating docks during a storm are substantial, especially on the fingers and at the juncture of the fingers and the main walkway. It is essential that special attention be directed to the joints and connections within the dock system. Design and construction that assures that the flotation devices (floats) stay connected to the dock structure during rough weather is needed.

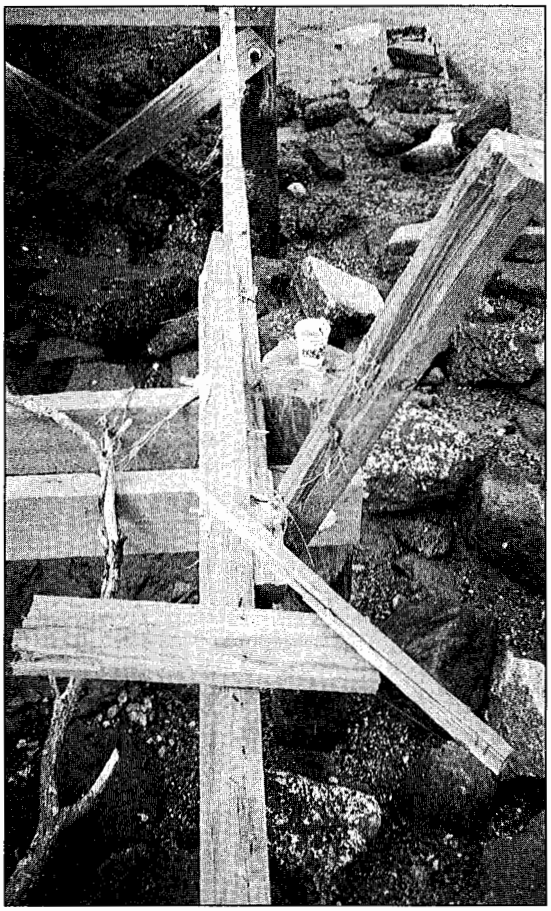

FIGURE 3. The decking and handrails fail first while the split pile cap and cross bracing have remained intact.

Retaining Walls, Stone Rip Rap & Other Beach Front Structures

Retaining walls on the beach front sustained severe damage from the storm surge and wave action. The Grand Strand area (Pawleys Island,

FIGURE 4. Floating docks were torn apart after tearing away from anchor piles.

Garden City, Surfside Beach, Myrtle Beach and North Myrtle Beach) is located between 75 to 110 miles northeast of Charleston along the coast (see Figure 1). Even these areas received a 12-foot tidal surge that caused substantial damage to ocean front hotels, businesses and residences. Structures that were protected by stone rip rap (one-man rip rap, with 30 to 150 pounds per stone) suffered less damage.

Similar structures near the storm's center (at Isle of Palms, Sullivans Island and Folly Beach) were destroyed by the combination of wind-driven waves and the storm surge. Even heavy stone rip rap structures such as groins on Folly Beach were damaged or destroyed. In all areas that had sand dunes, the damage was minimized because the dunes were sacrificial to the structures behind them. Once dunes were swept away, the structures behind the dunes (retaining walls, rip rap, swimming pools and building foundations) took the remaining force of waves, wind and rising waters, and failed (see Figure 5).

Timber sheet pile retaining walls on the beach sheared off at the ground line either from rising tides and surf, or outgoing tides that had washed the backfill and anchors from behind the retaining walls. Returns on the retaining walls helped, but in some cases the front retaining wall was torn away with the returns left intact. The coastal structures behind the beach, outside the Federal Emergency Management Agency (FEMA) velocity zone, exhibited little damage if they were designed and constructed with suitable, stronger materials, adequate tie backs and end returns.

Recommendations. The first line of defense for beach front structures is good planning. Locating habitable and commercial structures behind the high hazard zone is economical and effective. The next most effective method of protecting structures is to enhance and improve sand dune restoration and maintenance. In small storms and lunar high tides, this type of protection will be adequate. In larger storms, dune destruction will be sacrificial, but will help in protecting subsequent structures such as retaining walls and rip rap. Stone rip rap can be valuable for attenuating wave forces; however, if small size stones are used, they can

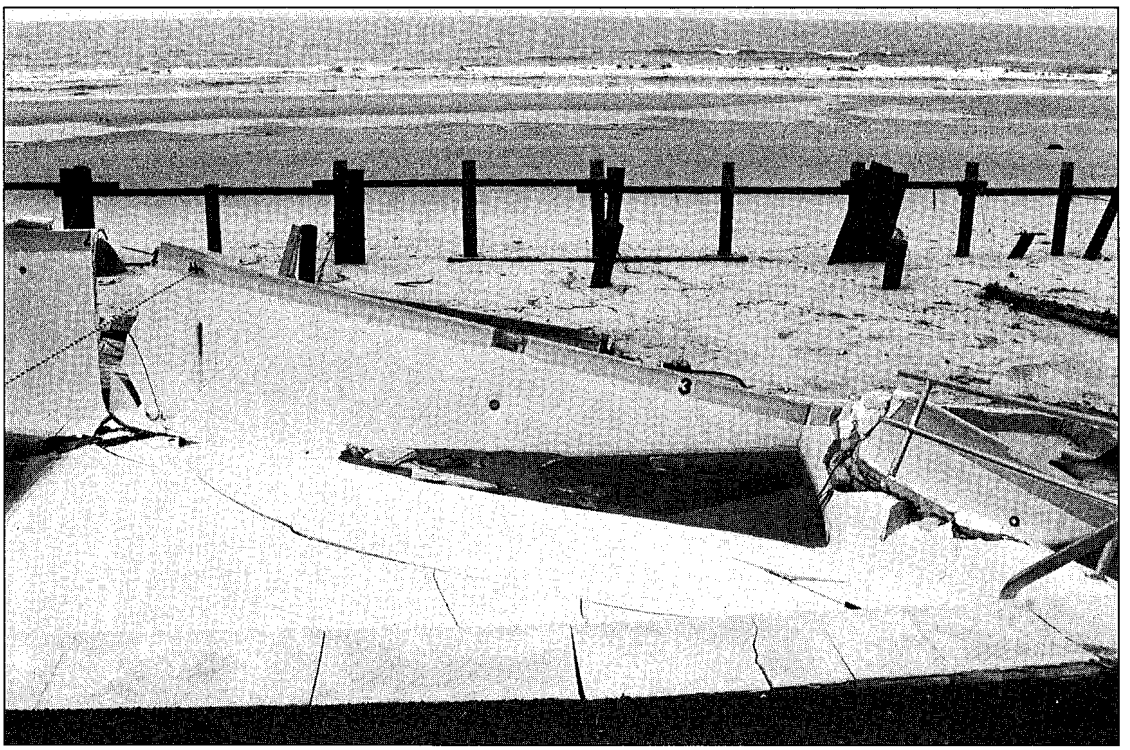

FIGURE 5. Sand dunes were washed away, exposing retaining walls and pools to the storm surge.

become missiles that can break glass, doors, *etc.*, during the storm. The rip rap should consist of minimum 200-pound each dense armor stone (granite) in conjunction with a geotextile membrane layer and gravel bedding between the heavy armor stone and the geotextile membrane. Where possible, retaining walls should not be utilized as the primary protection. However, if retaining walls are used, the toe should be protected with stone rip rap. If timber retaining walls are used on the beach front, the minimum thickness of sheets should be three inches. There should be returns on the ends of the structure when the retaining wall does not connect to the adjacent property. Special efforts should be made to design the structure in conjunction with the protection available on adjacent property, since the failure of an adjacent neighbor's structure may suddenly mean that the structure could be exposed from a different direction.

Fishing Piers

Pier fishing is very popular on the Grand Strand beach area (northern Georgetown County and all of Horry County) and in the southeastern beaches of Brunswick County, North Carolina. Although these areas were from 65 to 150 miles from Hugo's path (center), substantial damage to fishing piers occurred. The storm surge in Myrtle Beach (100 miles from Charleston) reached 12 feet. Most of the fishing piers were constructed of timber and all were over ten years old. Many had been repaired after past hurricanes, but still utilized older construction methods. The only fishing pier directly in the path of the eye of Hurricane Hugo was the Breach Inlet Bridge fishing pier located between Sullivans Island and Isle of Palms. This timber structure was mounted on the side of the bridge, but it was torn from its anchors in the bridge and was lifted by the tidal surge and damaged beyond repair. The remaining structure has been removed and may be replaced by a free standing structure soon. Many fishing piers were torn from their piling supports by the combination of uplift from submergence and horizontal forces from surf, drag

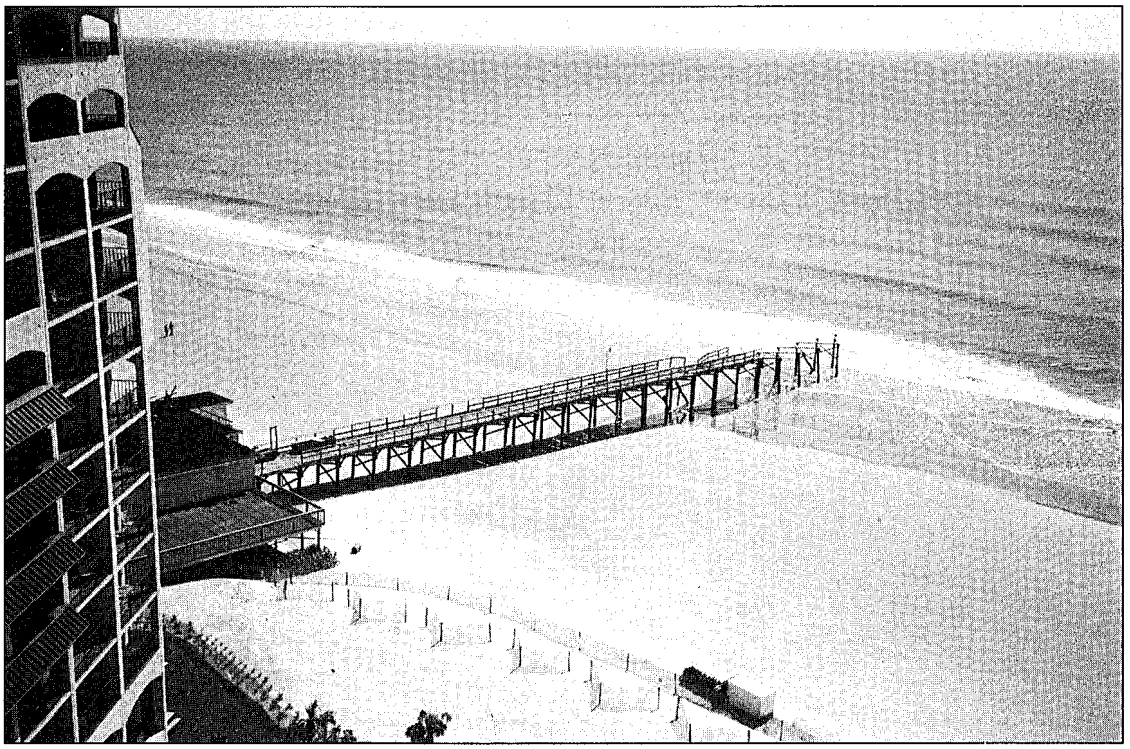

FIGURE 6. Sections of this fishing pier near shore sustained less damage. At the seaward end of the pier, pinned pile caps lifted off with the deck structure, thereby exposing piles and cross members to drag forces during the storm surge.

and wind. Sections near the shore (the first 300 to 400 feet) where pile length exposure (length from the ground line to the bottom of the pile cap) was less did not sustain as much damage as did sections in deeper water (see Figure 6).

The pile caps on many of these fishing piers were pinned through the top of the pile cap into the top of the pile with steel drift pins. The uplift forces lifted the pile cap and deck structure off, leaving piles with no top bracing. Once piles on the fishing piers lost their pile cap integrity, the cross bracing was not adequate to keep the piles intact. Scour at the base of piles exacerbated the problems by increasing the unsupported length of pile. Piles then came out intact, broke in the surf, leaned over or broke at the mud line. Once piles and deck structures were free in the surf, they continued inland to cause more damage to shore structures. From piles that washed ashore, it appears that some piles broke at midspan, probably from drag. The piles on fishing piers had cross bracing just below the deck structure. There were no cross braces below that point. The pile pairs were cross braced in the transverse direction and were not cross braced in the longitudinal direction. There was little evidence of the use of spiked grid connectors on the cross bracing to pile connection. Many of the galvanized bolts in the intertidal and splash zone had deteriorated (rusted).

Recommendations. There are no existing codes that are applicable to fishing piers. Only references to handrails and live loads are applicable from current building codes (Standard Building Code). Since these structures serve the public, their design should be dictated by professional engineers utilizing state or local agency mandated minimum design standards for materials and performance.

Fishing piers should be designed for a hierarchy of failure as follows:

1. Handrails and decking.
2. Stringers.
3. Pile caps.

FIGURE 7. A three-foot tidal surge and hurricane-force winds ripped out fixed-pier facilities in Fajardo, Puerto Rico.

4. Cross bracing.
5. Piles.

The handrail and decking failure still leaves a structure that can be made serviceable very quickly and economically. Once pile caps and cross bracing fail, it takes a major effort and outlay of funds to make the structure serviceable again. The minimum bolt size utilized in fishing piers should be 0.75 inches. Spiked grids should be used in the cross bracing and the split cap pile connections to the piles. Connector straps should be utilized if pinned pile cap connections are used. Metal connectors used in the intertidal and splash zones should be coated with coal tar epoxy after installation.

Puerto Rico & the Virgin Islands

Hurricane Hugo's path passed just west of St. Croix, Virgin Islands, then just east of Fajardo, Puerto Rico, on its northward advance toward the United States mainland. Hugo's sustained winds have been estimated at 125 miles per hour in St. Croix and 82 miles per hour at San Juan International Airport.[4] While there are no storm observations available from the Caribbean, the SLOSH Model (the National Weather Service storm surge computer program) predicted storm surge water levels of three to four feet above normal at St. Croix and the eastern end of Puerto Rico.[5]

Because of the small daily tidal fluctuation, marinas in Puerto Rico utilized fixed piers only. Many of the marina facilities are old and had been modified utilizing piles from existing structures. Hugo's wind and tidal surge ripped out many fixed piers that were located in exposed locations (see Figure 7). Structures — piles, abutments and fixed piers — that had been modified from the original construction often failed because the increased loads on the structures exceeded the original design. Many of the older concrete structures showed excessive deterioration (rust) in steel reinforcing and exposed metal. Recently constructed marinas that utilized professional design and modern materials survived Hugo with little damage.

Recommendations. Fixed-pier marina facili-

ties should incorporate design considerations for the uplift that results from wind and storm surge. If the boats are to remain in the marina, fixed piers and piles should be designed for impact and anchoring live loads. Older marina structures should be reevaluated for their reliability in future similar hurricane events. Additions to older existing fixed-pier and pile structures should be constructed only when designed by professional engineers.

Reconstruction of Destroyed Coastal Marine Structures

In South Carolina, the docks, especially floating docks, have been upgraded. However, for the most part, the piles in floating dock systems are not being sufficiently upgraded to meet similar hurricanes in the future. Residential fixed piers and floating dock systems have not been upgraded. In the case of fixed piers and fishing piers, some have been upgraded, some have not been rebuilt, and on some the old mistakes are being repeated by the use of inadequate piles and pinned pile caps. Retaining walls and erosion control (rip rap), when permitted, have been reinstalled as before. The "beach retreat" policy of the South Carolina Coastal Council does not allow for the improvement of the structures that were destroyed. "One man" rip rap stone (135 to 150 pounds) has been used on many beach front projects. Contractor designs for replacement structures have also been used. Neither insurance companies, lending institutions nor permitting agencies have required professional involvement in many of these projects.

Conclusions

It is difficult to establish, on a case by case basis, the exact failure mode of structures during a hurricane. The combination of forces is varied and difficult to predict and does not necessarily approximate a set of formulas or a laboratory experiment. Only by looking at a number of installations with various types of construction can patterns of failures be developed that are conclusive and that should be translated into design standards and/or codes. Some design features should be obvious and historical in their application. However, with the advent of new materials, as well as the prevalence of design personnel with little experience, inexperienced marine contractors and developers that are trying to construct marina projects as economically as possible, the need for guidance in marina and coastal structure design has never been more apparent. ASCE Manual 50 *Report On Small Craft Harbors,* was published in 1969 and has not been updated since that time.[6] There is a committee that is active on a revision, but nothing is promised in the immediate future. A key design reference, "Small Craft Harbors: Design, Construction and Operations," by James W. Dunham and Arnold A. Finn, was published in 1974 and has not been revised since.[7] These factors — combined with a growing need to protect boaters, marina investors, marina insurers, financial institutions, adjacent property and the public at large — make the development of modern design standards a top priority.

Assuming that a very good set of design standards exists that could be used by competent marina designers, there are many marina projects that never have any involvement by registered design professionals. Many of these projects utilize designs developed by contractors or dock builders/manufacturers. Often, building permits and inspections by building officials are not even required on marina projects. If required, the inspectors often do not know what to look for; they have no code requirements to enforce. Insurance companies are insuring marinas without even knowing what standard of design was used in the facility. The marina insurers were hit hard by Hugo. Rates for insurance will likely be changing. Some insurance companies are beginning to give preferred rates to marinas that utilize design professionals. Lending institutions, likewise, have shown little concern for the design or the performance potential of marinas that are financed by them.

Environmental and resource agencies are reviewing marina permit applications with only a narrow view for their particular agency's consideration and little concern for navigation, boater safety, economic feasibility or public access to the water. The location, design, construction and operation of marina facilities is now being affected by these agencies that have little knowledge about the technical or opera-

tional aspects of marinas and coastal structures.

Engineers must recognize that structural adequacy and considerations for materials are no longer the only marina and coastal structure planning and design criteria. Because of increased public interest and environmental concerns, there are new considerations beyond the normal engineering purvey. It is time that total marina planning and design requirements be taken into account and a broad-based design standard that integrates most of these concerns should be developed. It is now obvious that this effort cannot come from only the professional design community or from government agencies. If engineers do not initiate positive leadership and action on marina standards and codes soon, the opportunity to control or affect the standards will be lost. In addition to the normal engineering design criteria, considerations for boater safety, marina operations, insurance, financing, international boating concerns and public safety should be addressed. The following factors are some that could be addressed in a new design standard:

1. Minimum hurricane or storm criteria for marinas should be developed. Adequate estimates of the winds and tides for different intensities of hurricanes can be obtained without much difficulty. The criteria should be developed around an expected return period for storms (*e.g.*, one in 50-year storm) occurring at the worst conditions (an astronomic high tide). If a marina is designed for this protection level, all parties — boaters, marina operators, *etc.* — will know how to react to an impending storm of a specified intensity.

2. Too much marina design is performed by contractors and dock builders who do not address all of the complex issues in the total marina design effort. In order to protect the public, government agencies need to set minimum marina planning and design criteria and need to require registered design professional involvement in marinas. (Maryland has currently instituted a program to develop a document that would satisfy this requirement for marina design and operation in areas that are subject to hurricanes.)

3. Marina design is not effective if the operation of the marina is not consistent with the design assumptions. If a marina designer assumes that boats will be evacuated from a marina in a hurricane and they are not (or they cannot be evacuated), then everything in the marina is at risk because of the erroneous design/operation assumption. If a marina owner cannot show a hurricane evacuation plan that demonstrates that the boats can be evacuated, the marina should be designed to hold the boats during the storm. Hurricane Hugo proved that complete evacuation of marinas is not possible.

4. Hurricane Hugo cost the insurers of marinas substantial money in claims. Indications are that the underwriters will be stricter on design or will withdraw from insuring marina facilities. Insurance companies and financing institutions have a responsibility to protect their investors and stockholders. The concerns for design life, design protection, and the amount of risk allocated to the boat in the marina and the marina facility should be made known to the design professional so marina designs that meet these requirements can be developed.

5. An internationally recognized standard for rating marinas is needed. International boaters need to know what level of service they can expect, what level of protection from the elements they can get, what additional services are available and what size craft can be accommodated. This rating system would encourage marina developers to address and assess the market better before committing to the project. Once a decision is made on the type of marina project they need to construct, the marina developer could then provide more realistic guidance about the project to marina planners and engineers.

6. State and local governments have a responsibility to provide leadership in setting minimum marina and coastal structures' codes. This goal may not be possible to accomplish on a nationwide basis because of the individual market and site-specific requirements for marina construction. Some elements that could be addressed in a marina design standards and construc-

tion code are as follows:

- Minimum hurricane/tornado design storm frequency (*e.g.*, one in 50-year storm).
- Design compliance certification by a registered professional engineer.
- Construction compliance certification by a registered professional engineer.
- Minimum piles sizes, and timber preservative treatment, if applicable.
- Minimum pile heights above mean sea level.
- Requirements for soils investigations and professionally engineered pile design to include pile penetration.
- Minimum loads for dock flotation, handrails and piers.
- Minimum floating dock freeboard under dead load and under combined dead and live load.
- Acceptable flotation materials for floating docks.
- Minimum gangway live loads, deflection criteria and angle at low tide.
- Minimum handicap design requirements.
- Minimum bolt and nail sizes.
- Minimum timber sizes and grades.
- Minimum corrosion treatment of construction materials.
- Minimum electrical requirements.
- Minimum water supply requirements.
- Minimum sanitary pump out requirements.
- Minimum parking requirements.

Summary

Hurricane Hugo probably caused more property damage than any storm in history (the estimate of damage is at 9 billion dollars). However, it is also the best documented storm in history. The lessons from Hurricane Hugo should not be forgotten. South Carolina residents were lucky — the state leadership evacuated areas in a timely manner. A more densely populated area may not be able to act so effectively. Engineers now have the knowledge and the ability to design for the next hurricane disaster. However, in the interest of their profession and the public, they need to secure the aid of building officials, government agencies, insurers and legislators in developing and implementing new codes and standards for marina and coastal structure design.

NOTE — *The author visited Puerto Rico in conjunction with a joint post-disaster evaluation by the International Marina Institute and the Puerto Rico Sea Grant College Program at the University of Puerto Rico.*

JON GUERRY TAYLOR *is founder and president of Jon Guerry Taylor, P.E., Inc., located in Mt. Pleasant, South Carolina. The firm provides engineering, planning and environmental consulting services for marina and waterfront projects. Taylor received his B.S. in Civil Engineering from the University of South Carolina in 1965. He is a Registered Professional Engineer in Florida, Georgia, South Carolina, North Carolina, Virginia and Michigan. He is a member of the International Marina Institute (IMI), the States Organization for Boating Access (SOBA), the Association of Conservation Engineers (ACE) and the American Society of Civil Engineers (ASCE).*

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The Mixing Zone for Combined Sewer Overflows: Testing the Concept as a Basis for Regulation

Models and approaches to applying the concept of mixing zones to the coastal zone require further refinement in order to meet water quality standards for resource areas.

THOMAS HRUBY

Under the provisions of current federal and state clean water acts, discharges of pollutants into coastal and inland waters in the United States are to be controlled or eliminated. To achieve this goal, state and federal agencies have been empowered to regulate both point and nonpoint source discharges. The regulations that are being promulgated are based on a wide range of criteria: some are linked to technological solutions, some to biological toxicity and some to the water quality conditions in the receiving water. One concept now being developed that is integral to the receiving water criterion is that of a *mixing zone.* The mixing zone, also called a *dilution zone,* is defined as a "limited area or volume of water where initial discharge takes place; and where numeric water quality criteria can be exceeded but acutely toxic conditions prevented from occurring."[1]

The concept of mixing zone has been further refined in policy and guidance documents by specifying additional constraints. These constraints may include ones that stipulate that mixing zones will not be allowed to extend into areas with critical resources, and/or it shall be small enough to allow adequate passage for motile organisms.

Initially, the concept of mixing zones was developed for regulating discharges into rivers and streams, and individual states have been empowered to set their own criteria for the size of the mixing zone. At present, 30 states have regulations for discharges into rivers, streams

or lakes based on a mixing zone. Recently, however, the mixing zone concept is also being considered as a criterion for discharges into coastal areas. As of 1988, the states of Florida and Hawaii, and the District of Columbia, had formalized mixing zone criteria for estuaries and coastal areas.[2]

A significant source of pollution to the coastal zone are discharges that occur from combined sewer overflows (CSOs).[3-5] These discharges occur because many municipalities, especially those along the eastern seaboard, have sewer systems that also collect storm runoff. Most of these sewer systems cannot carry the combined flows of sewage and stormwater during heavy rains, and the excess is discharged into the nearest body of water, the coastal zone in this instance. Now that the direct discharges from wastewater treatment plants are in the process of being controlled in the United States, CSO discharges are coming under closer regulatory scrutiny.

Since the mixing zone concept is currently in use for discharges into lakes and rivers, it is also being considered as a basis for regulating CSO discharges into the coastal zone. This approach is attractive from an engineering point of view because it would reduce the need for expensive technical solutions at the point of discharge by, *de facto,* lowering the water quality standards in the immediate vicinity of the discharge, as long as coastal resources are not impacted. Dilution by the receiving waters would become part of the treatment process for meeting water quality criteria.

The use of the mixing zone concept in regulating CSOs, however, assumes that a method or a model for estimating the size of the mixing zone exists, and that the water quality standards will actually be met within a reasonable distance of the discharge point. It is further assumed that the mixing zone will not extend into nearby resource areas. These assumptions, however, need to be tested in the coastal zone before the approach becomes widely accepted as a basis for drafting regulations. If analyses show that water quality standards are not consistently met in resource areas, then the mixing zone concept would not be useful as a basis for determining regulations.

Initial studies have indicated that discharge plumes from CSOs can travel large distances without adequate mixing,[6] and there is a concern that the aforementioned basic assumptions are not usually met. The plume of a CSO discharge will almost always impact a nearby beach, shellfish bed or natural resource before it is adequately diluted to meet water quality standards.

The study described here was undertaken to determine whether the mixing zone concept provides any flexibility in regulating CSO discharges in two coastal areas. If the size of the mixing zone always includes resource areas, then the concept would not provide any flexibility in the design of a mitigation approach. The regulatory decision would be that no discharge is permitted. The results of a dye study in New Bedford, Massachusetts, to assess the size of the mixing zone from a CSO discharge, and attempts to use models for determining the size for CSO discharges in Lynn, Massachusetts, are reported here.

Measuring Mixing Zones in New Bedford

During the period between February 15 and May 12, 1989, a series of six dye studies at five of the 38 combined sewer overflows in New Bedford were performed as part of a CSO facilities plan.[7] Only the results from two batch release dye studies done at one CSO (Number 004 at the north end of Clarks Cove — see Figure 1 for its location) are reported here, since that CSO provided the best test of the mixing zone concept. The other four dye studies were complicated by technical problems such as a plugged CSO, the termination of flow and discharge under docks, which prohibited establishing an accurate picture of the mixing zone.

The details of the methods used for the dye studies are described in the CSO facilities plan,[7] and only a summary is presented here. Approximately ten liters of a 20 percent aqueous solution of Rhodamine B dye were dumped into the CSO discharge. The CSO discharge was pumped through the hurricane barrier at a rate of 55,000 gallons per minute, and a total of approximately 600,000 gallons were discharged during each of the two studies.

The dispersion and dilution of the dye released into the CSO discharge was mapped for

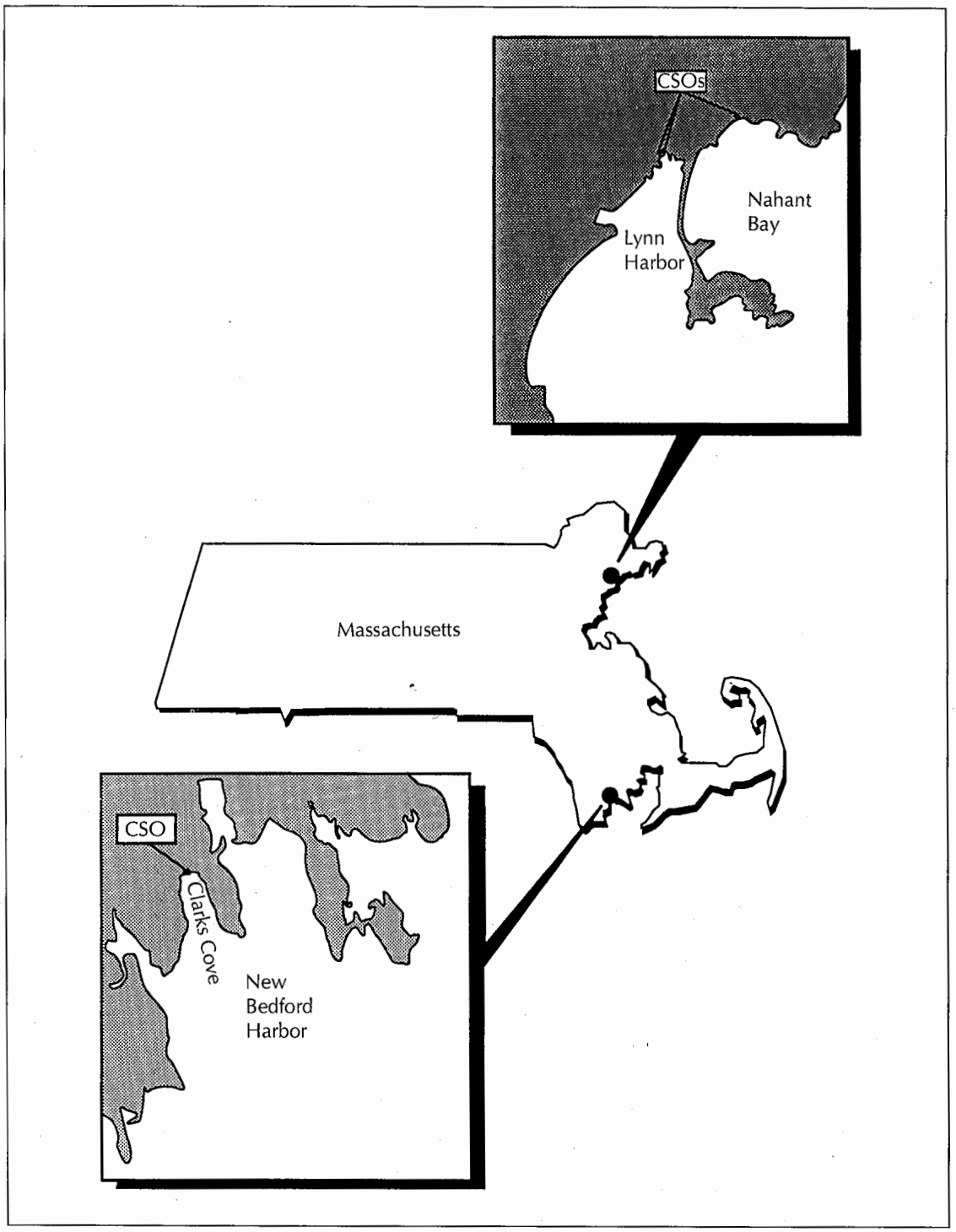

FIGURE 1. Location of CSOs in Lynn and New Bedford, Massachusetts.

six hours using a flow-through fluorometer mounted on a vessel. Horizontal positioning of the survey vessel was provided by using an electronic positioning system. Dye concentrations and the size of the plume were mapped at hourly intervals by recording dye concentra-

tions along perpendicular transects that crossed the plume. Vertical profiles of dye concentration, salinity and temperature were also performed at regular intervals along the transects.

Modeling the Mixing Zone

The mixing zone of discharges from waste treatment plants is often estimated by modeling the dilution and dispersion of the discharge plume in the receiving water. With outfalls located offshore, in deep water, two-dimensional models are often sufficiently accurate to be used.[8] The major factors that influence the mixing zone are the relative buoyancy of the discharge and the velocity of the currents in the area.

In coastal waters, however, two-dimensional models do not provide enough detail to completely predict the dilution and transport of a CSO discharge. Factors such as wind-induced surface currents, tidal eddies, the density difference between a surface discharge of freshwater and the underlying salt water all interact to affect the mixing. Furthermore, most models available at present use a deterministic approach in order to model pollutant transport. CSO discharges, however, are stochastic, and the pollutant mixing and transport is not easily characterized by deterministic models.[9]

Some of these problems can be overcome by developing three-dimensional deterministic models that can incorporate most factors. Such models exist, or are under development, but they are extremely complicated and expensive because they attempt to model stochastic events using a deterministic approach. Their cost quickly becomes prohibitive given the number of CSOs that need to be modeled in coastal areas.

Because two-dimensional models do not completely characterize the three-dimensional nature of the mixing from a freshwater CSO discharge into saline coastal waters, four different models were used to provide an estimate of the size of the mixing zone for three CSOs in Lynn (the locations of the CSOs modeled are shown in Figure 1). The first model simply considers dilution based on the volume of the discharge and the volume of the receiving water. The second and third models are two-dimensional estuarine models, and the fourth is a density flow model. None of these models accurately describe all aspects of mixing when a CSO discharges because each model takes into account only a few of the many factors that influence plumes. The models were used together, however, in order to outline the scope of the problem and to provide a preliminary estimate of the size of the mixing zone.

Dilution Model. A first-order estimate of the size of the mixing zone can be made by determining the volume of the receiving waters that is needed in order to provide the level of dilution required to meet water quality standards, based on the pollutant loadings in the discharge. The area represented by the volumes of water needed to dilute discharges from different storms can be plotted and areas where the mixing zone impinges on important resources can be identified.

Estuarine Flow Models. A dynamic estuarine model (DEM) was used to predict the size of the mixing zone in Lynn Harbor, while the Tidal Embayment Analysis/Eulerian Lagrangian Analysis model (TEA/ELA) was used in Nahant Bay. Both of these models incorporate tidal flows and the intermittent nature of the discharge in the modeling. Both models can address wind effects, albeit crudely. They do not, however, include the density differences between the discharge and the ambient water. The description of the models and their use for the Lynn discharges is given in one section of the facilities plan.[10] Briefly, the models were calibrated with oceanographic data that were collected in the field, and then the discharge from the two-week and five-year storms were modeled using average tidal conditions.

Density Flow Model. The two estuarine models used, DEM and TEA/ELA, do not take into account any stratification that may occur. This aspect of the mixing dynamics was investigated using an expert system computer program for mixing zone analyses of waste discharges.[11,12] The expert system predicts the dilution that can be achieved for submerged discharges that have a different density than the receiving water. The model that is derived from this analysis takes into account density differences between the discharge and the receiving water, the velocity of the discharge and

the local currents that can move the plume. It does not, however, model any wind-induced mixing that may occur.

Results of the Dye Studies in New Bedford

The procedure for estimating the mixing zone from the dye studies was to determine the estimated dilution achieved in the plume as a function of time and next to map the distance traveled by the plume in that time. A detailed description of the results and their analysis is given in the final facilities plan.[7] Figure 2 presents a graphical summary of the path of the dye plume on April 20, and the concentrations of dye at different times. On March 30, the plume traveled to the western shore and was contained within the shallow waters within one kilometer of the discharge.

By relating the peak concentration of dye with the time since discharge, it is possible to estimate the dilution rate of the discharge. Figure 3 presents the dilution of dye as a function of time for the two studies. The dilution rate initially seems to be linear. In the first study (on March 30), the dilution rate was 2.5/hour *(i.e.,* the concentration of dye decreased by a factor of 2.5/hour). In the second study, the dilution rate was 10/hour. The significant difference in dilution rates can be attributed to the observation that on March 30 the plume was pushed against the western shore of the bay and held there by the wind. On April 20, the plume was moved along the shore, and in waters that were 20 feet deep, thus permitting better mixing.

The distance traveled by the plumes was most closely correlated with wind direction and velocity. On the average, the plume moved at approximately one to 2.5 percent of the wind speed as measured at the New Bedford Hurricane Barrier, depending on the proximity to shore. Tidal currents in Clarks Cove are very small, and the average tidal excursion is less than one kilometer.[7]

Based on these observed dilution rates, it was possible to estimate the time that would be needed for the CSO discharge to meet water quality standards. The average concentrations of the two most concentrated pollutants (measured during the field studies undertaken for the facilities plan), copper and coliform bacteria, were utilized in order to obtain estimated mixing times.

Initially, the mean concentration of fecal coliform bacteria in the discharge was 156,000/100 milliliter (ml), and that of copper was 0.11 milligram/liter (mg/l).[7] Based on a dilution rate of between 2.5 to 10 per hour and assuming that the 90 percent of the bacteria die within 24 hours (T_{90} of 24 hours), it is estimated that the beach standard for total coliform in Massachusetts (200 fecal/100 ml) would be met in 21 to 32 hours. The shellfish standard for fecal coliform, which is stricter (14 fecal/100 ml), would be met in 42 to 54 hours. Copper, which is assumed to be a conservative pollutant that remains in the water column, will take four to 16 hours to reach water quality standards (2.9 μg/l).

Shellfish beds are found throughout Clarks Cove in New Bedford Harbor,[13] and beaches are found along both shores of Clarks Cove. During the dye studies, the discharge plume reached the beaches within four hours of discharge, and were over the shellfish beds almost immediately. Given the time needed to meet water quality standards for coliform bacteria, the mixing zone *(i.e.,* the area where water quality standards are not met) extended into the resource areas. Based on a detailed analysis of 40 years of wind data, it was found that water quality standards for coliform bacteria will be violated throughout most of Clarks Cove over 30 percent of the time a discharge occurs through this CSO.[7]

Modeling

The four aforementioned models were used to estimate the size of the mixing zone for CSOs in Lynn. Of the six major CSOs in Lynn, the discharges from three were analyzed: two that discharge into Lynn Harbor and are combined in the calculations, and one that discharges into Nahant Bay (see Figures 1 and 4). The flow characteristics from these CSOs are summarized in Table 1, and are based on the data collected for the Lynn CSO facilities plan.[10] The marine resources that can be impacted from the CSO discharges are summarized in Figure 4. These resources include shellfish beds, beaches and lobstering areas.

Dilution Model. Using National Oceano-

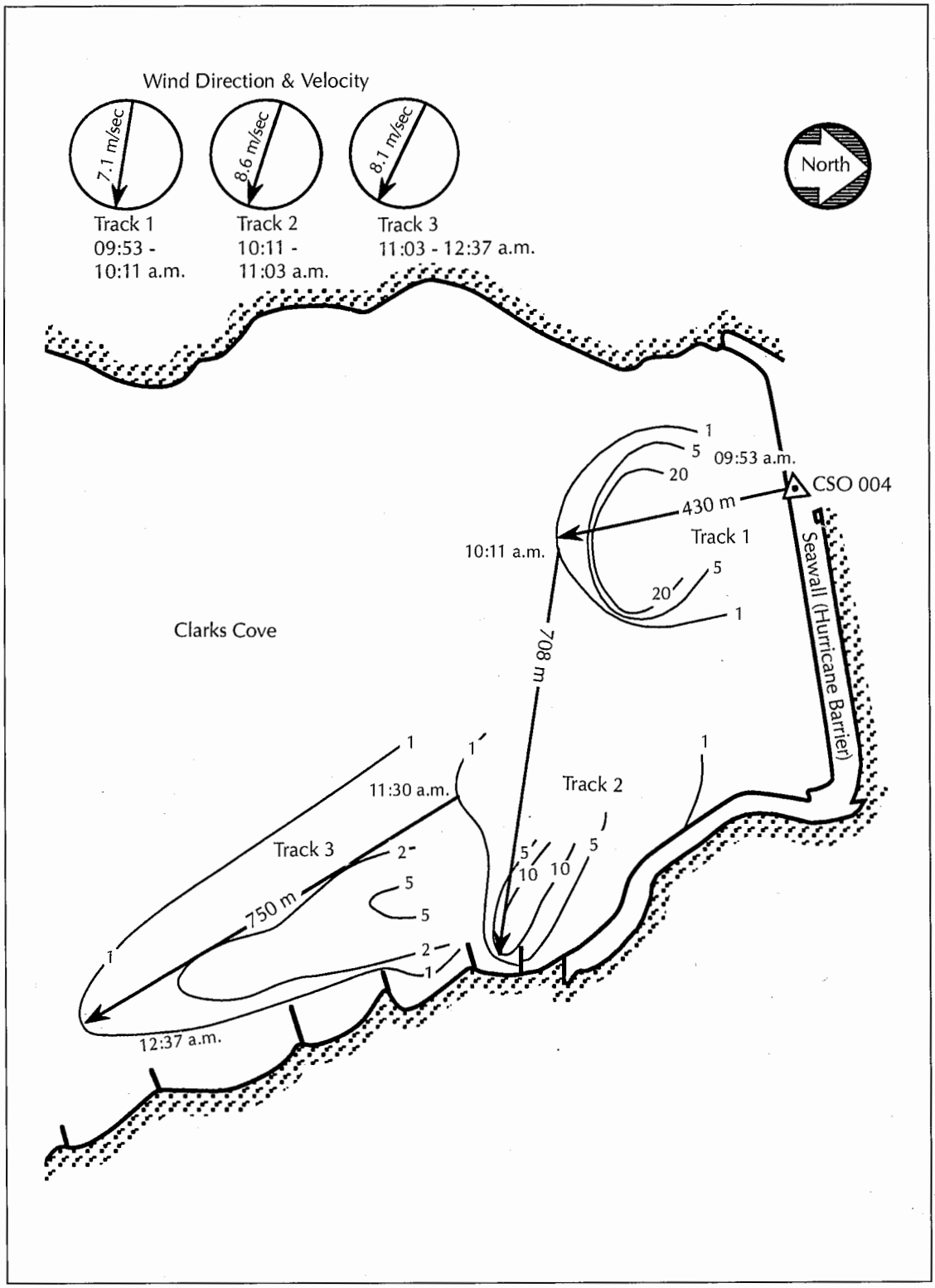

FIGURE 2. Leading edge of dye plume from a CSO discharge on April 20, 1989, in Clarks Cove, New Bedford Harbor. Contours represent dye concentrations in parts per billion.

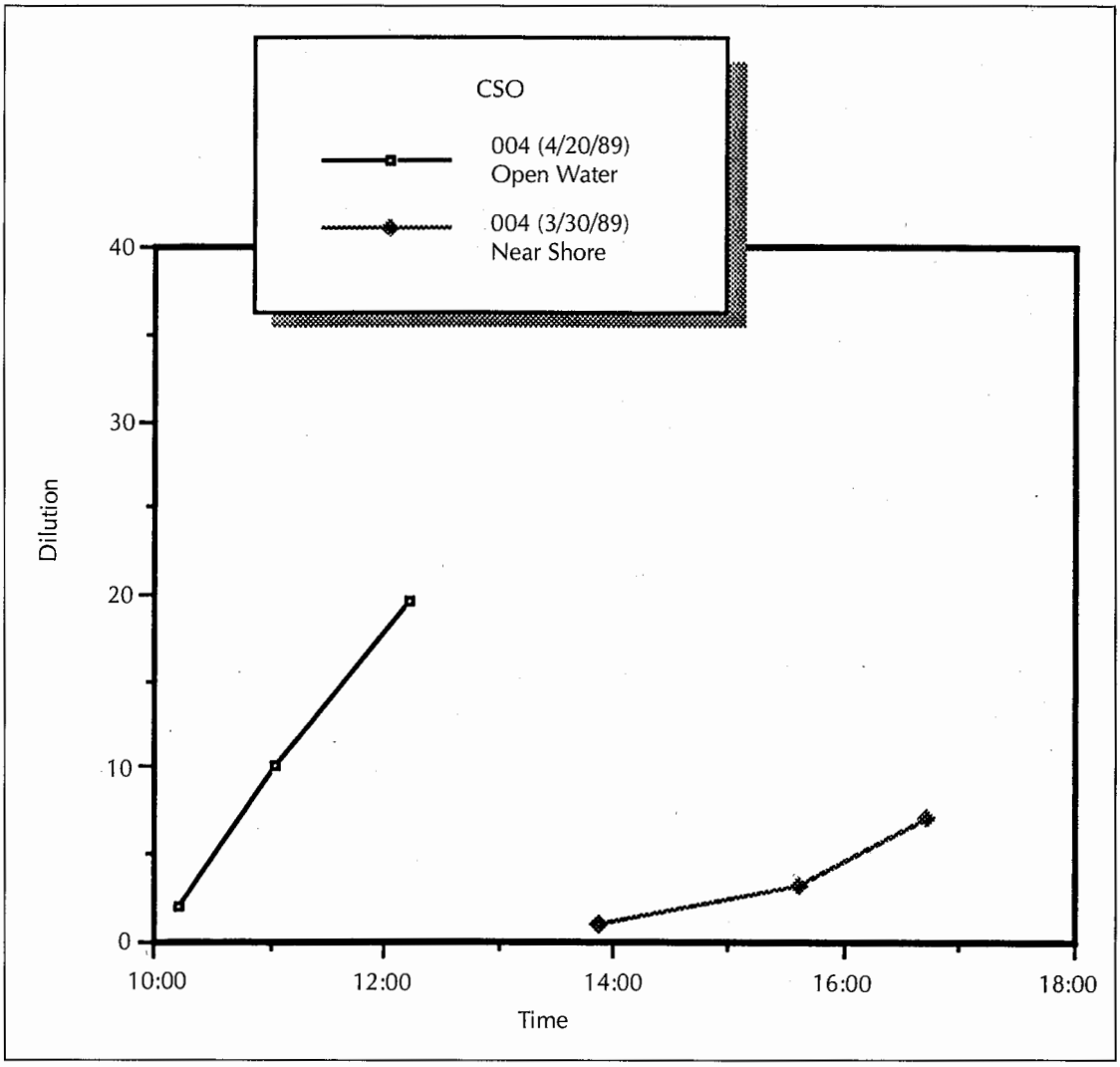

FIGURE 3. Dilution of CSO discharge in New Bedford Harbor as a function of time.

graphic and Atmosphere Administration (NOAA) chart #13275 of Lynn Harbor and Nahant Bay, it is estimated that there are approximately 150 million gallons of water in Lynn Harbor at low tide in the area extending between the north end of the harbor and its mouth (a line running east of the Nahant Peninsula). In Nahant Bay, the estimated volume is 19,000 million gallons between the two points that span the bay.

Copper and coliform bacteria are two pollutants needing the highest dilution in the CSO discharges into Lynn Harbor. The average fecal coliform count is 200,000/100 ml and the copper concentration is 60 parts per billion (ppb).[10] To achieve the state shellfish standard of 14/100 ml, therefore, a dilution of approximately 14,000 is needed. If the less rigorous standard of 88/100 ml is used in areas for restricted shellfishing, a dilution of approximately 2,300 is needed. To achieve the U.S. Environmental Protection Agency (EPA) water quality standard of 2.9 ppb for copper,[14] a dilution of approximately 20 to one is required.

With a two-week storm discharging 0.6 million gallons, and a five-year storm discharging 11.7 million gallons through the two CSOs, volumes of 1,380 and 26,900 million gallons, respectively, are needed to dilute the discharge in order to meet the less stringent coliform stan-

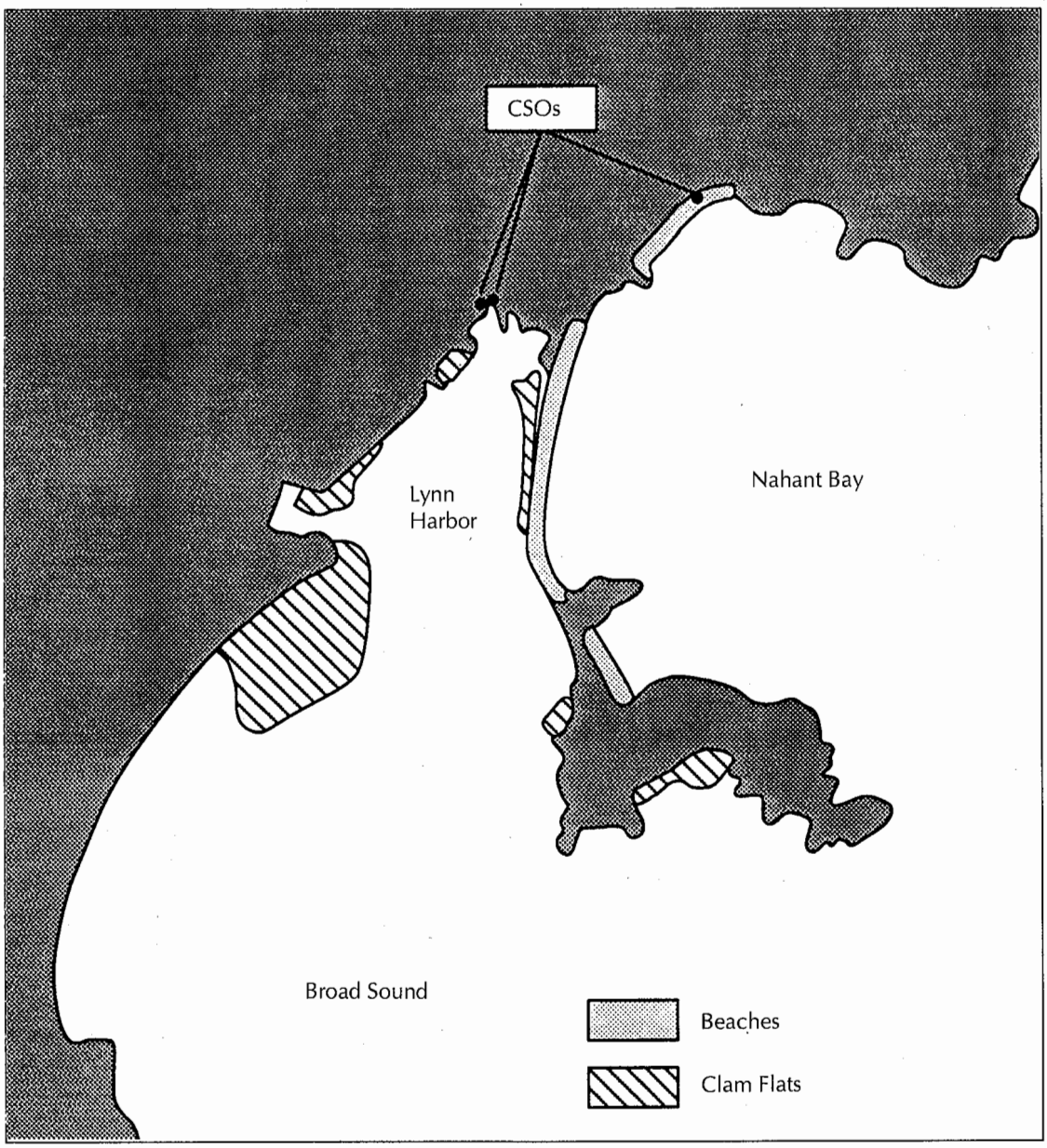

FIGURE 4. Lynn Harbor and Nahant Bay showing the locations of CSOs and critical resources that may be impacted by the discharges.

dards. For copper discharges, the volumes of the receiving waters needed are 12 and 234 million gallons, respectively, for the two storm conditions.

The CSO discharging into Nahant Bay has fecal coliform concentrations of approximately 260,000/100 ml, but it does not have any measurable levels of copper.[10] Since there are no shellfish beds in Nahant Bay, the only water quality standard for coliform bacteria that needs to be met is that for beaches (200/100 ml). Thus, a dilution of 1,300 to one is needed in the bay in order to meet the standard. By comparing the estimated size of the receiving waters with the dilutions needed to meet water quality criteria, neither the harbor nor the bay is large enough to adequately dilute the discharges from a five-year storm, and Lynn Harbor is

TABLE 1
Characteristics of CSO Discharges into Lynn Harbor and Nahant Bay

Location of CSO Discharge	Average Flow 2-Year Storm (Million Gallons)	Average Flow 5-Year Storm (Million Gallons)	Dilution Needed to Meet Minimum Coliform Standard*	Dilution Needed to Meet Copper Standard*
Nahant Bay	2.3	42.0	1,300:1	none
Lynn Harbor	0.6	11.7	2,300:1	20:1

* The dilutions noted are those needed to meet water quality standards based on the concentrations of the pollutants as measured in the discharge.

even too small to adequately dilute coliform bacteria during a two-week storm.

This estimate is crude, since it does not take into account the duration of a storm nor the tidal flushing that takes place. It can be used, however, in order to provide a computationally simple first-order approximation of the mixing zone where water quality standards are not met from the CSO discharges. The comparison of the potential mixing area with the resources in the area (compare Figures 4 and 5) shows that resources are impacted by the discharges, even if the estimate is wrong by a factor of two or more. Thus, the mixing zones of the CSOs in Lynn Harbor and Nahant Bay as they are estimated from a "dilution" model will almost always impact some critical resource.

Estuarine Flow Models. The results from the DEM and TEA/LEA models, summarized in Figure 5, show that by adding a time factor, the dilution of the effluent is improved relative to the simpler model described previously. The figure illustrates the dilutions 24 hours after an overflow began. The results from these models, however, still indicate that water quality criteria would not be met by the time the plume from a five-year storm reaches critical resources. In Nahant Bay, the plume reaches the beaches before a 1,300 to one dilution is achieved. In Lynn Harbor, shellfish beds are subject to a plume that is diluted only by a factor of 100 to one, not the 2,300 to one needed in order to meet the criteria.

Density Flow Model. Since coastal CSOs discharge freshwater into salt water, the density difference between the two may result in the formation of a lens of effluent that does not mix immediately with the receiving waters. Measurements of dispersion in actual plumes have shown that the density differences may significantly enlarge the mixing zone. In Jamaica Bay, New York, plumes of effluent were still measurable more than one mile from the discharge point.[6]

In order to use the expert system, which requires submerged discharges, the site of the CSO discharge in Nahant Bay was changed to a hypothetical offshore location at a depth of five meters. When the conditions for a two-week storm were modeled, using the field measurements of ambient currents, temperature and salinity,[10] the model indicated that the plume would extend over two kilometers before a dilution of 100 to one is achieved. These results again suggest that the discharge would not be mixed well enough in order to meet water quality criteria at the beaches.

Conclusions

There are several key factors that extend the size of the mixing zone in coastal areas:

- The relatively small volume of receiving waters;
- Strong tidal currents;
- Wind; and,
- The density difference between discharged waters and the receiving waters.

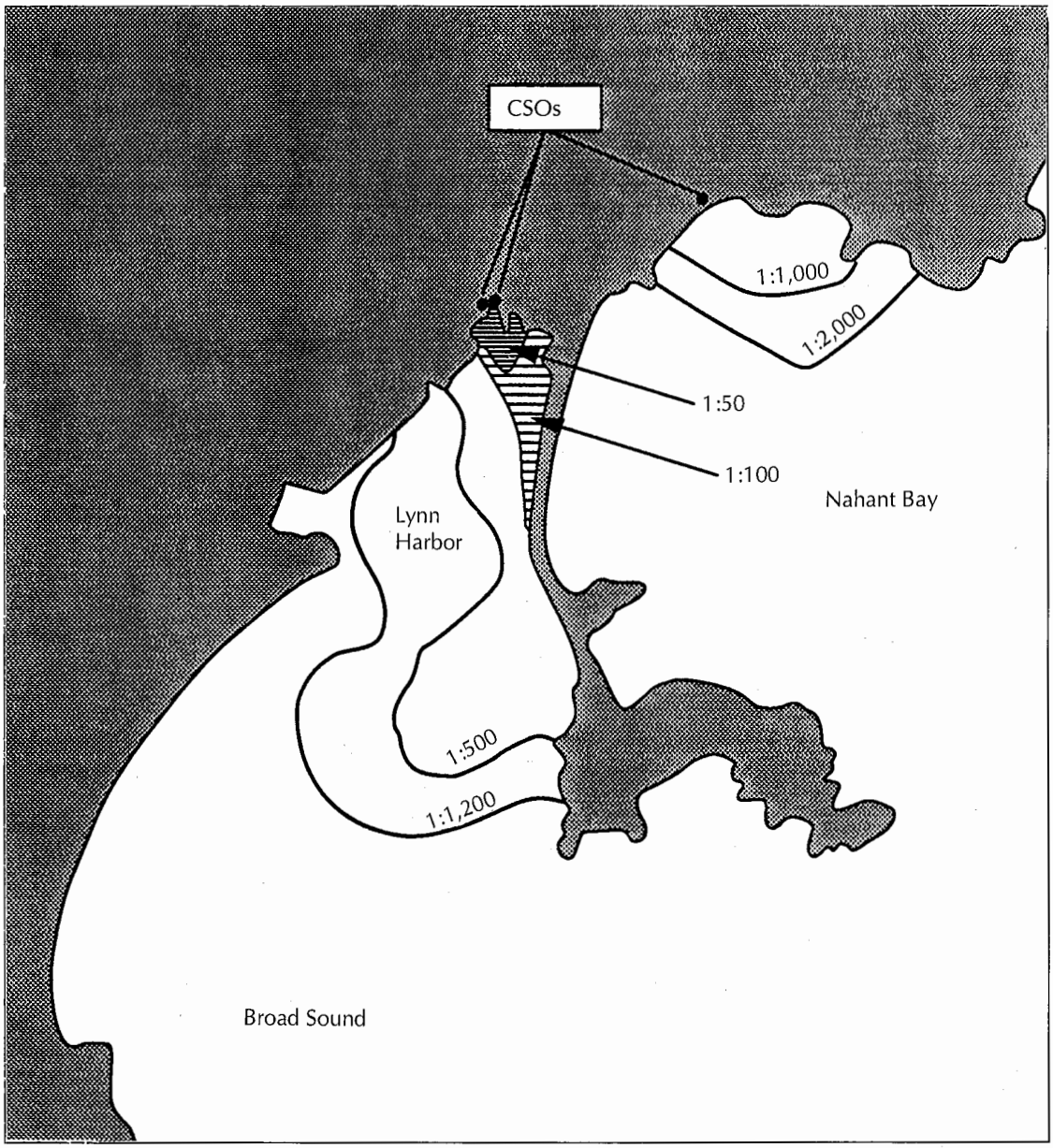

FIGURE 5. Dilutions for a five-year storm as predicted by DEM and TEA/LEA for Lynn Harbor and Nahant Bay.

In the example of the CSO discharge in New Bedford, the mixing zone extended well beyond the nearest beaches and shellfish beds. In Lynn, all four modeling approaches indicated that the mixing of discharges would not be sufficient in order to meet water quality standards for nearby resources. Since the other coastal communities in the Northeast such as Boston, New Bedford, Salem and Gloucester, all have CSOs discharging into shallow constricted bays, similar problems can be expected in these cities. The mixing zone of the discharge would extend to nearby resource areas for these communities.

All of these observations suggest that the concept of mixing zone for regulatory purposes may not be very useful for CSO discharges into coastal areas. Important resources will almost

always be within the zone where water quality standards are not met, and the basic regulatory conclusion would be that no CSO discharges are to be permitted. Therefore, regulations based on the mixing zone concept in the coastal zone would not provide the needed flexibility for developing the appropriate remediation that would meet both environmental and economic needs.

The size of a mixing zone can be reduced by some form of treatment before discharge. Such an assumption was not made in this analysis because only the concept of mixing zones as a general approach to regulations was explored. Treatment at the point of discharge becomes a very site specific problem because each effluent has different pollutant characteristics and has different amounts of dilution available in the receiving waters.

If the concept of mixing zones is not readily applicable to CSO discharges, do other options for developing regulations exist? One idea being considered by regulators in Massachusetts is to limit the number of discharges. The assumption is that every discharge from a CSO would cause a water quality violation and impact some resource. The impacts on the coastal environment would be reduced by limiting the discharges to only a few per year. This approach makes technical control measures more feasible, since there is no need to design systems to handle extreme storm conditions that occur infrequently. The unresolved question with this approach is whether a few "pollution" events per year is an acceptable environmental compromise.

CSO regulations are still in their embryonic stage, and now is the time to explore different concepts. It is hoped that the summary of these experiences in trying to apply the concept of the "mixing zone" in New Bedford and Lynn will stimulate discussion of the issue. In the future, there is a need to explore other concepts in more detail, including the feasibility of using the "permitted violations" concept. In addition, there is a need to develop an empirical model for assessing the size of CSO plumes in different coastal environments.

ACKNOWLEDGMENTS — *The data for this study were collected as part of CSO facilities plans for the cities of New Bedford and Lynn, Massachusetts. Oceans Systems, Inc., was the subcontractor for the dye studies. The DEM was developed by Camp Dresser & McKee (CDM) and the TEA/ELA model was developed by the Parsons Laboratory at the Massachusetts Institute of Technology. Most of the modeling using TEA/ELA and DEM was performed by Mitchell Heineman and Robert Kapner of CDM. Joanne Barker of CDM did the volume estimates of Lynn Harbor and the CSO loadings. A Turner Model 10 fluorometer and the Racal "Micro-Fix" electronic positioning system were employed in the New Bedford site study. CORMIX1 was the expert system used for analyzing the mixing zone.*

THOMAS HRUBY *is a Senior Scientist at Camp Dresser & McKee, Inc. (CDM). He joined the firm in 1988 and specializes in analyzing coastal, marine and wetland ecosystems. His work has included innovative methods and models to assess the impacts of nutrients in coastal water and developing regional coastal monitoring programs. Prior to joining CDM, he worked for eight years as a senior scientist with the Massachusetts Audubon Society, and two years as a post-doctoral fellow in Marine Policy and Ocean Management at the Woods Hole Oceanographic Institution. He holds a B.A. in biochemistry from Harvard University, an M.Sc. in oceanography from the University of Washington and a Ph.D. in botany from the University of Glasgow in Scotland.*

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The Old Colony Railroad Rehabilitation Project

Developing a transportation plan involves reviewing alternatives to determine which best meet transportation needs, are technically & environmentally sound & reflect community goals.

DOMENIC E. D'ERAMO &
RODOLFO MARTÍNEZ

Southeastern Massachusetts has been the state's fastest growing area in population over the past decade. This growth, particularly that of the working-age population, has led to increased congestion on existing highways and transit facilities servicing this area. Continuing residential growth in Southeastern Massachusetts communities, combined with commercial development in downtown Boston, has resulted in an ever-increasing travel demand that will be impossible to satisfy. Unlike other parts of the metropolitan area, there are no adequate or attractive travel alternatives to the private automobile for many commuters who are faced with these increasingly congested facilities.

The Massachusetts Bay Transportation Authority (MBTA) and the Urban Mass Transportation Administration (UMTA) have undertaken the Old Colony Railroad Rehabilitation Project to improve transportation services to Southeastern Massachusetts. This project has involved an extensive alternatives study for improving transportation services in the region. The alternatives involving the restoration of commuter rail service make use of four existing railroad rights-of-way — the Main Line, the Middleborough Line, the Plymouth Line and the Greenbush Line. All of these lines, some 80 miles in total length, were part of the former Old Colony Railroad system that ran through Southeastern Massachusetts (see Figure 1).

Following a scoping and screening process, in compliance with state and federal environmental regulations, the MBTA and UMTA completed and circulated a Draft Environmental Impact Statement/Report (DEIS/DEIR) in May 1990. In the DEIS/DEIR, six alternatives were selected and studied in detail:

1. No-Build.

2. Transportation Systems Management (TSM).

3a. Commuter Rail from South Station to Middleborough.

3b. Commuter Rail from South Station to Middleborough and Plymouth.

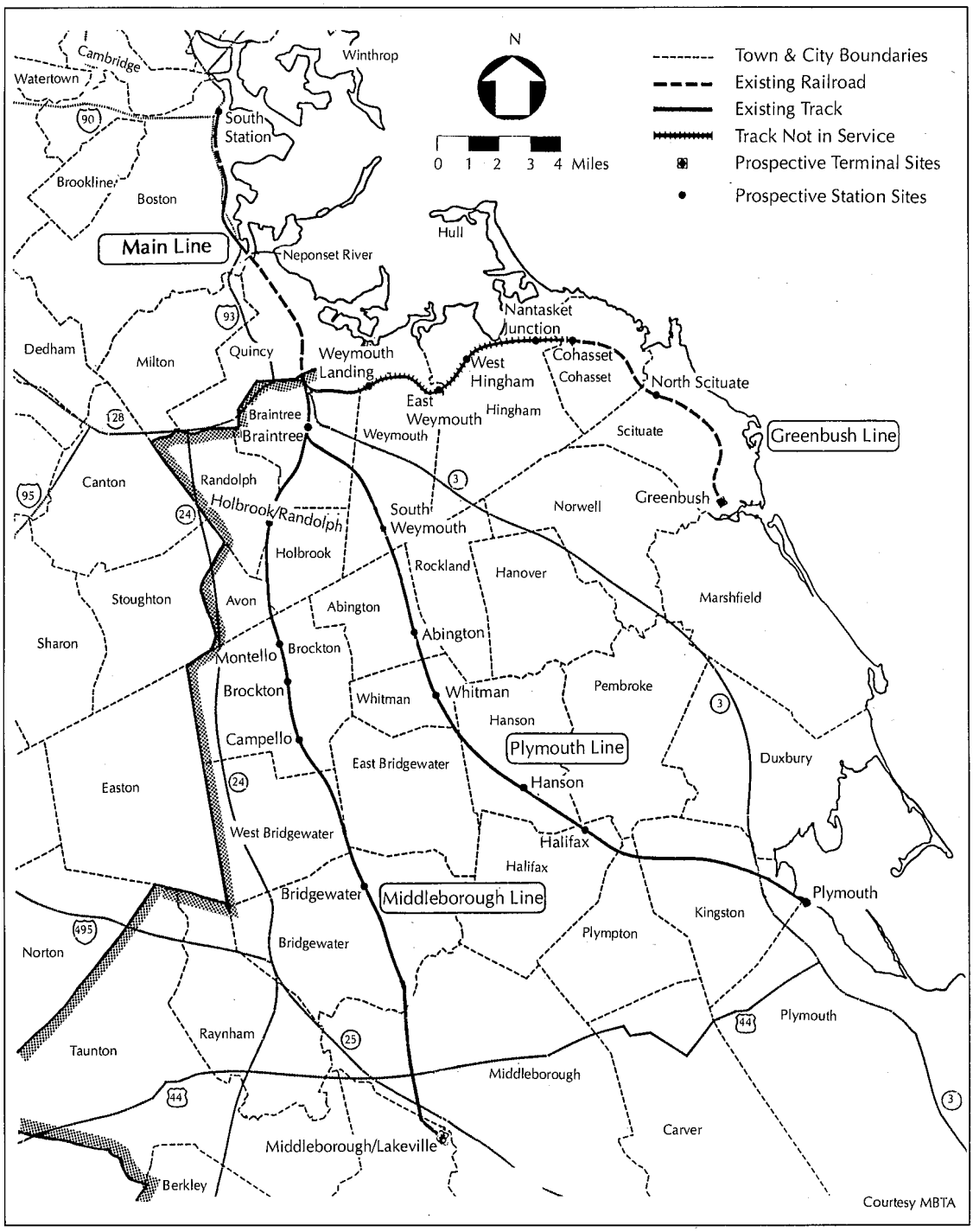


FIGURE 1. Old Colony study area and railroad system.

3c. Commuter Rail from South Station to Middleborough and the Greenbush area of Scituate.

3d. Commuter Rail from South Station to Middleborough, Plymouth and Greenbush.

After an extensive public review and comment period that lasted until the fall of 1990, the

MBTA and UMTA decided to proceed with project development by preparing a Final Environmental Impact Statement/Report (FEIS/FEIR) for the restoration of commuter rail service on the Main Line, the Middleborough Line, and the Plymouth Line, a modified version of Alternative 3b. This project involves the restoration of almost 60 miles of railroad rights-of-way, the construction of a 1,200-foot-long bridge over the Neponset River, and the construction of 14 new railroad stations and four park/ride lots. The preparation of the FEIS/FEIR has been completed and circulation of the document will take place this fall.

Restoration of commuter rail service on the Middleborough and Plymouth lines does not address the transportation needs of the Greenbush Line corridor. To respond to comments reflecting opinions that the alternatives involving restored commuter rail service on the Greenbush Line may have significant impacts on historic resources near the existing rights-of-way, UMTA requested that the MBTA undertake a Section 4(f) Evaluation for some historic sites in Hingham, including the Lincoln National Register District which abuts the Greenbush Line rights-of-way. In this Section 4(f) Evaluation, the MBTA will study reasonable and prudent alternatives to the restoration of commuter rail service on the Greenbush Line and will consider all reasonable measures to avoid use of, and minimize harm to, protected historic sites. The Section 4(f) Evaluation for the Greenbush Line is expected to be finished by the end of 1991.

The Project Area

Study Area. The Old Colony study area includes 32 communities extending south from Braintree towards the Cape Cod Canal and Buzzards Bay and west from Massachusetts Bay to Route 24 (see Figure 1). These communities, along with Quincy and Boston, represent the area that would be affected by the transportation alternatives under review. The study area encompasses approximately 450 square miles and includes heavily urbanized areas such as Brockton and Quincy, some large suburban towns such as Weymouth and Braintree, and many areas with a rural character and developable land.

The "Old Colony" designation for the area refers to its origins as the first British colony in the northeast, established at Plymouth in 1620. The Massachusetts Bay Colony, established in Boston somewhat later in 1630, grew larger than the Plymouth settlement and eventually incorporated the older colony. The term "Old Colony" was widely used during the nineteenth century, and was adopted by an early railroad company that provided passenger and freight service in the area.

Many of the long-established Old Colony study area communities are now experiencing, and are projected to continue to experience, population and development growth. In 1980, the total population of the study area was approximately 600,000 people. By the year 2000, the study area population is projected to increase by about 100,000 people. This projected growth rate is expected to exceed that for the Boston metropolitan area as a whole by as much as five to ten percent. The Old Colony area work force has a strong orientation to Boston. A substantially higher proportion of the work force commutes to Boston and Cambridge for work than the work force from north and west of Boston. Sixteen percent of total work trips from the Old Colony area are to Boston and Cambridge, compared to eight percent from the northern communities and twelve percent from the western communities.

History of the Old Colony Railroad Lines. Passenger railroad service was established throughout the Boston metropolitan area in the mid-nineteenth century, including extensive service in southeastern Massachusetts. In November 1845, the Old Colony Railroad established a passenger line from Boston to Plymouth. Service to Middleborough and beyond to Fall River was provided in 1846 by the Fall River Railroad, which merged with the Old Colony in 1854. Passenger service was provided to Cohasset in 1849 and to Kingston by 1874 via the South Shore Railroad, which also became part of the Old Colony system in 1877.

By 1893, when the Old Colony Railroad became part of the New York, New Haven and Hartford system, the Old Colony included a

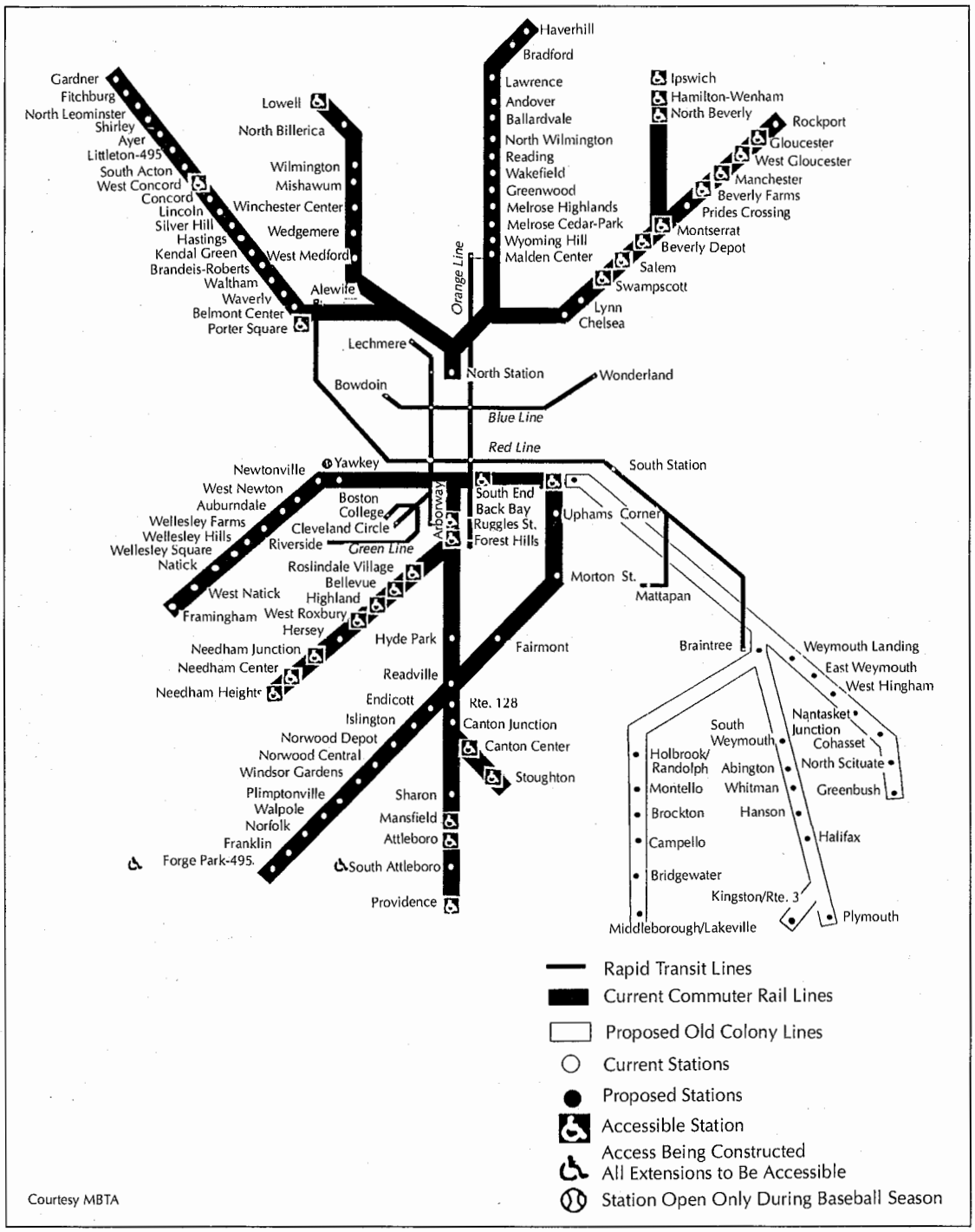


FIGURE 2. MBTA commuter rail system.

large network of rail lines extending from Boston, Lowell and Fitchburg to Cape Cod and Providence. Double-tracked lines extended as far as Greenbush on what is now known as the Greenbush Line, as far as Whitman on the Plymouth Line and beyond Middleborough on the Middleborough Line. Four tracks were in operation from South Station in

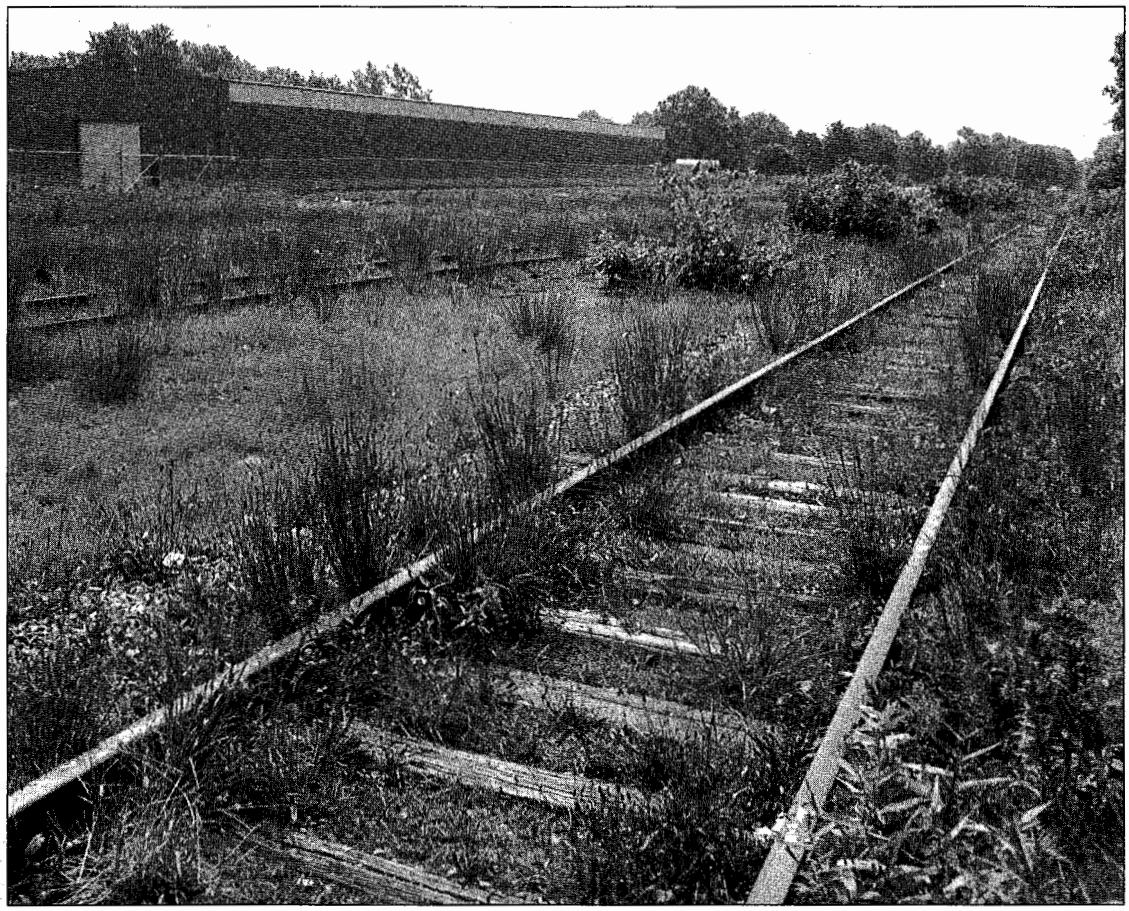


FIGURE 3. A view of the existing Plymouth line track in Abington. The project includes rights-of-way improvements to modern commuter rail standards allowing up to 60 miles per hour passenger service.

Boston as far as North Quincy.

Passenger rail service and ridership reached a peak in the metropolitan Boston area in the early 1900s. However, faced with increasing competition from automobiles, diesel buses and electrified rapid transit lines and streetcars, railroad service throughout the metropolitan area declined after World War I. Most of the second track on the Old Colony lines was removed by 1941. World War II's fuel rationing and employment boom resulted in a temporary increase in ridership, but major service cuts in 1949 reflected the long term loss of ridership. By 1956, the New York, New Haven and Hartford Railroad was eager to discontinue what it felt was a very unprofitable passenger service on the Old Colony system. Court orders and a one-year state subsidy kept service operating until June 30, 1959, when passenger service finally ceased on the Greenbush, Plymouth and Middleborough lines.

While those three Old Colony lines were the first to go, a similar fate threatened the other commuter railroad lines serving Boston. In July 1959, the Massachusetts legislature created the Mass Transit Commission to study this and other transportation problems. In the early 1960s, on the recommendation of the commission, the state sponsored service experiments to determine whether commuter rail and express bus ridership would respond to service improvements and fare reductions. When the MBTA was formed in 1964, it began a comprehensive region-wide program of subsidizing passenger rail service. Unfortunately, by this time a 1960 fire had destroyed the Old

Colony line railroad bridge over the Neponset River making experimental service on the Old Colony Lines physically impossible. Any consideration of Old Colony commuter rail service was further de-emphasized when the MBTA extended Red Line rapid transit service from Boston to Quincy in 1971 and further south to Braintree in 1980.

With MBTA ownership and capital investments, the Boston area commuter rail system has shown substantial improvement in terms of service levels, equipment condition and reliability, and ridership. Today, commuter rail plays an important role in the transportation network in all parts of the metropolitan area except the Old Colony study area (see Figure 2). Parts of the Old Colony rights-of-way have been in limited use for freight service over the intervening years, but in general they have been a dormant, vastly underutilized resource from the time commuter rail service stopped in 1959 (see Figure 3).

The Purpose & Need for the Rail Lines

Transportation Issues in the Study Area. Area highway and transit facilities do not meet future, or even existing, needs for access to Boston. The study area experiences severe highway congestion daily. All Old Colony primary highways operate at a level of service (LOS) C or worse during peak periods. For the Southeast Expressway, the main limited access highway connecting Boston to the Old Colony study area, LOS F (force flow conditions) last for four hours in the morning and for three hours in the afternoon. Based on traffic growth trends, Routes 3 and 24, the two limited access highways that feed into the Southeast Expressway, are expected to operate at LOS E by the year 2000. Parking facilities for commuters in downtown Boston and at transit stations are at capacity and opportunities for expanding these facilities are limited.

MBTA Red Line rapid transit service from Boston through Quincy to Braintree runs frequently during peak hours, but it extends only into the northernmost portion of the study area. It operates at or near capacity during peak hours. Parking lots and garages at stations are regularly filled to capacity on weekdays prior to 7:30 a.m. Private express bus service, existing MBTA feeder service, service provided by other transit agencies and private, non-profit vanpool company are also area transit options, but are subject to severe congestion on the highway system.

The commuter boat service between the Hingham Shipyard and the Boston waterfront provides an attractive commuting alternative, but its market coverage is limited to the immediate area of the shipyard. The opportunities for expanded boat services in the Old Colony area appear very limited. In this environment, transit has a difficult time competing with the private automobile.

The congestion on major highways and secondary roads serving the Old Colony area and the crowding on the Red Line leads to significant travel delays and inconvenience. This congestion has an economic cost, measured in the cost of moving goods or the productivity of employees, and reduces access opportunities for residents and employees in the area. Anticipated growth in travel demand will put further pressure on transportation facilities that are now taxed to the limit. The end result will be increasing isolation of the Old Colony area residents from downtown Boston employment, educational and cultural opportunities.

In all other parts of the metropolitan area, commuter rail has been restored and upgraded to the point where it provides an attractive and reliable alternative for commuting into Boston. The residents of the study area have no comparable option. This inequitable distribution of transportation service options is the most glaring and the most often cited shortcoming of the current transportation system in the Old Colony area.

The Origin of the Project Concept. The Old Colony Railroad Rehabilitation Project originated as part of the ongoing system planning process. System plans are regularly re-examined, and additional studies of particular issues, system elements or particular corridors may be conducted. In the Boston area, this system planning is usually done by the Central Transportation Planning Staff (CTPS). The current status of system-wide planning is described in the Regional Transportation Plan. Under federal guidelines, the plan is reviewed

TABLE 1
Locally Adopted Metropolitan Boston Transportation Goals & Objectives

Goal	Objective
Improve Transportation Services to Improve Mobility	Increase Transit Capacity Reduce Transit Travel Time Increase Transit Accessibility Increase Transit Reliability & Comfort Increase Transit Ridership Reduce Parking Demand
Provide Transit Services That Are Cost-Effective	Maximize Use & Capacity of Existing Facilities Undertake Careful Analysis of All Projects
Provide a More Equitable Distribution of Transportation	Increase Services for Regions Now Poorly Served or Underserved Increase Access for Disabled Individuals or Individuals with Special Needs

by the Joint Regional Transportation Committee (JRTC), and revised and endorsed annually by the designated Metropolitan Planning Organization (MPO).

These regional plans continue to express the local commitment to devote more thorough attention to the role of the commuter rail network as part of the entire transportation system. As a result, the Massachusetts Executive Office of Transportation and Construction (EOTC) and the MBTA prepared a Commuter Rail Improvement Program (CRIP) that proposed the phased upgrading of the existing commuter rail lines. This blueprint for action has resulted in a continuing rail modernization program, and a larger, more modern fleet, improved signals, rights-of-way and stations.

In the early 1980s, there was a growing local perception that these system plans did not adequately address the transportation needs of the South Shore. In particular, it was noted that the CRIP effort did not cover the South Shore despite the existence of the Old Colony rights-of-way. Several South Shore communities conducted non-binding referenda that revealed that there was support for restoring commuter rail service. In 1984, the Massachusetts legislature formally requested that the Secretary of Transportation and Construction re-examine the transportation plans for the South Shore area, and report in particular on the feasibility of restoring passenger rail service on the former Old Colony lines. The Old Colony Feasibility Study of 1984 established concepts and cost estimates for providing commuter rail or other guideway transit services in the Old Colony study area.

As a result of the feasibility study, the MBTA and EOTC, working with the state legislature and other agencies in the area, identified the Old Colony study area as appropriate to be added to the rail modernization program and initiated the environmental studies of this project in late 1985.

The Goals and Objectives of the Project. The Old Colony Railroad Rehabilitation Project is part of a comprehensive effort to achieve a series of broad study area transportation and development goals, as well as specific objectives for improving the quality of transportation services and the equity of the distribution of services within the study area. These locally adopted goals and objectives, summarized in Table 1, support a broad, long-term study area

TABLE 2
Alternatives Development Process

Alternative	Alternatives Considered in the Feasibility Study	Alternatives Proposed for Further Study as a Result of the Feasibility Study	Alternatives Considered as a Result of the Scoping Process & the Public Involvement Program	Alternatives Proposed for Further Study as a Result of the Alternatives Screening Process
No-Build	No-Build	No-Build	No-Build	No-Build
Transportation Systems Management (TMS)	Diesel Bus Through Service From South Station to End Terminals	Diesel Bus Through Service From South Station to End Terminals	Improvements to Existing Diesel Bus Service & New Park-&-Ride Lots	Improvements to Existing Diesel Bus Service & New Park-&-Ride Lots
	Diesel Bus Transfer Service From South Station to End Terminals	Diesel Bus Transfer Service South Station to End Terminals	Diesel Bus Service from Quincy Adams Station to Southampton St.	
	Red Line Service Improvements	Red Line Service Improvements		
Diesel Powered Locomotive	Diesel Powered Locomotive Through Service From South Station to End Terminals	Diesel Powered Locomotive Through Service From South Station to End Terminals	Diesel Powered Locomotive Through Service From South Station to Middleborough	Diesel Powered Locomotive Through Service From South Station to Middleborough
	Diesel Powered Locomotive Through Service From South Station to Intermediate Terminals	Diesel Powered Locomotive Through Service From South Station to Intermediate Terminals	Diesel Powered Locomotive Through Service From South Station to Middleborough & Plymouth	Diesel Powered Locomotive Through Service From South Station to Middleborough & Plymouth
	Diesel Powered Locomotive Transfer Service From Braintree Station to End Terminals	Diesel Powered Locomotive Transfer Service From Braintree Station to End Terminals	Diesel Powered Locomotive Transfer Service From South Station to Middleborough & Greenbush	Diesel Powered Locomotive Transfer Service From South Station to Middleborough & Greenbush
			Diesel Powered Locomotive Through Service From South Station to Middleborough, Plymouth & Greenbush	Diesel Powered Locomotive Through Service From South Station to Middleborough, Plymouth & Greenbush
Diesel Railcar	Diesel Railcar Through Service From South Station to End Terminals	Diesel Railcar Through Service From South Station to End Terminals		
	Diesel Railcar Through Service From South Station to Intermediate Terminals	Diesel Railcar Through Service From South Station to Intermediate Terminals		
	Diesel Railcar Transfer Service From Braintree Station to End Terminals	Diesel Railcar Transfer Service From Braintree Station to End Terminals		
Electric Powered Locomotive			Electric Powered Locomotive Service	
Trackless Trolley	Trackless Trolley Through Service From South Station to End Terminals			
	Trackless Trolley Transfer Service From Braintree Station to End Terminals			
Electrified Light Rail Vehiceles	Electrified Light Rail Vehicle Through Service From South Station to End Terminals			
	Electrified Light Rail Vehicle Transfer Service From Braintree Station to End Terminals			
Red Line			Extension of the Red Line Along a Rte. 3 Alignment	

development and transportation strategy. The basic elements of this strategy are:

- Maintain downtown Boston as a strong economic hub for the study area so that the metropolitan area can remain economically sound and prosperous.
- Encourage transit-oriented development patterns to reduce the negative impacts of automobile dependency such as increased needs for highways and parking facilities, reduced air quality, other undesirable environmental effects and urban sprawl.

The Old Colony study area has been selected as a high-priority corridor by EOTC because of the major deficiencies in its current transportation system. Specific transportation problems in the Old Colony area include:

- Lack of transportation capacity to serve downtown Boston.
- Severe congestion on highways and transit facilities serving the study area.
- Inequitable distribution of transportation benefits.

The Transportation Alternatives Considered

The Initial Scoping and Screening Processes. The Old Colony Project benefitted from extensive public involvement during its scoping and screening process and environmental review. A wide variety of interested citizens, as well as federal, state and local officials and agencies, regional agencies and other public organizations and community groups were involved in the public involvement program. The end result has been a thorough and comprehensive review of alternative transit improvements that may be possible in the Old Colony study area. Table 2 summarizes the alternatives development process. Eight alternatives were identified in the Old Colony Feasibility Study as worthy of further analysis. These alternatives were:

- No-Build, in which all existing commuter transportation modes would continue to operate as they do now or with committed improvements.
- TSM, consisting of improvements to express bus service to Boston, feeder bus service to the Red Line and the Red Line itself.
- Commuter rail through service from South Station to end terminals at Middleborough, Plymouth and the Greenbush area of Scituate.
- Diesel rail car through service from South Station to end terminals at Middleborough, Plymouth and Greenbush.
- Commuter rail through service from South Station to intermediate terminals at Campello, either Whitman or Hanson, and West Hingham.
- Diesel rail car through service from South Station to intermediate terminals at Campello, either Whitman or Hanson, and West Hingham.
- Commuter rail transfer service from the Braintree Red Line station to terminals at Campello, either Whitman or Hanson, and West Hingham.
- Diesel rail car transfer service from the Braintree Red Line station to terminals at Campello, either Whitman or Hanson, and West Hingham.

Alternatives considered and rejected during the feasibility study were trackless trolleys on paved Old Colony rights-of-way and electrified light rail vehicles (LRVs) on existing Old Colony rights-of-way. While these alternatives have the advantage of reduced air and noise impacts when compared with commuter rail and diesel rail car technologies, they would include elements incompatible with existing commuter rail facilities and services, would impose undue constraints on rail freight operations and would require higher capital costs.

Several alternatives in addition to the eight proposed in the Old Colony Feasibility Study were introduced for consideration during the scoping and screening process. These additional alternatives were:

- Electrification of commuter rail service.
- Buses on paved Old Colony rights-of-way.

- Extension of the Red Line rapid transit along a Route 3 alignment.
- Sub-options under commuter rail through service making use of different combinations of lines.

As the scoping and screening processes progressed, capital and operating costs and benefits (quantified as market area coverage, ridership and travel time savings), engineering implications (such as physical feasibility, compatibility with existing facilities and services, and impact on freight operations), general environmental concerns and community inputs resulted in dropping several of the alternatives. Among those alternatives that were dropped were proposals for:

- The use of diesel rail cars.
- Commuter rail to intermediate terminals.
- Transfer to the Red Line rapid transit in Braintree.
- Electrification of commuter rail service.
- Buses on paved Old Colony rights-of-way.
- Extension of the Red Line rapid transit along a Route 3 alignment.

The No-Build, the TSM and the commuter rail sub-options were retained and subjected to detailed analysis in the DEIS/DEIR.

Alternatives Evaluated in Detail in the DEIS/DEIR. Six alternatives were examined in detail in the DEIS/DEIR (see Figure 4). These alternatives were:

1. No-Build. No further transportation improvements would be made beyond those now funded or committed.

2. TSM. Improvements to express bus service to Boston in order to achieve shorter peak period headways and reduce travel time and construction of 15 additional park-ride lots.

3a, 3b, 3c & 3d. Restoration of commuter rail service, with locomotive powered push-pull commuter rail train operations from South Station to terminals in Middleborough/Lakeville, Plymouth and Greenbush in Scituate. Four combinations of commuter rail service were considered. Alternative 3a included commuter rail service only on the Middleborough Line. Alternative 3b had service on the Middleborough

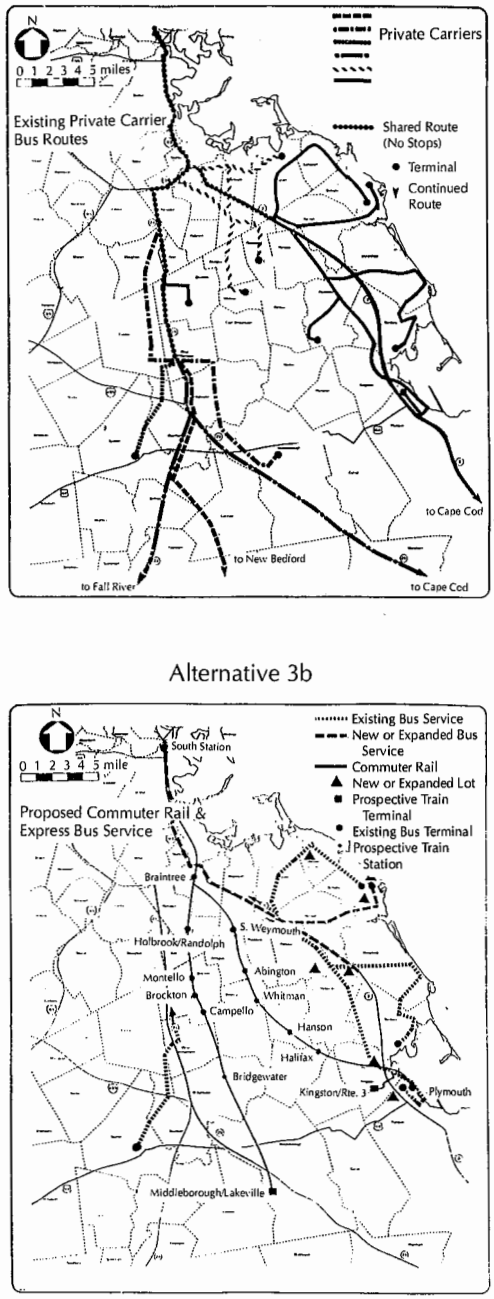
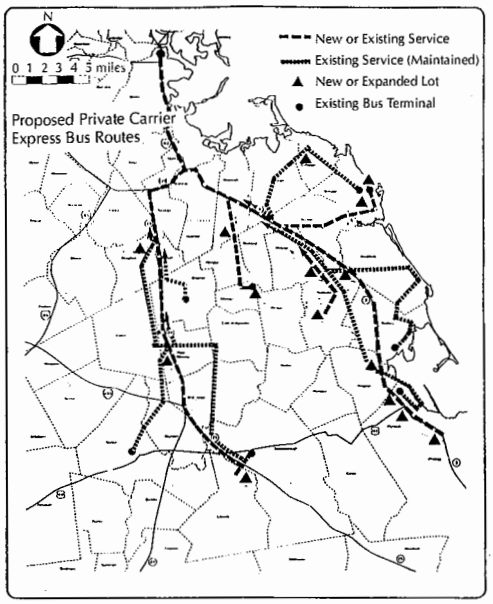
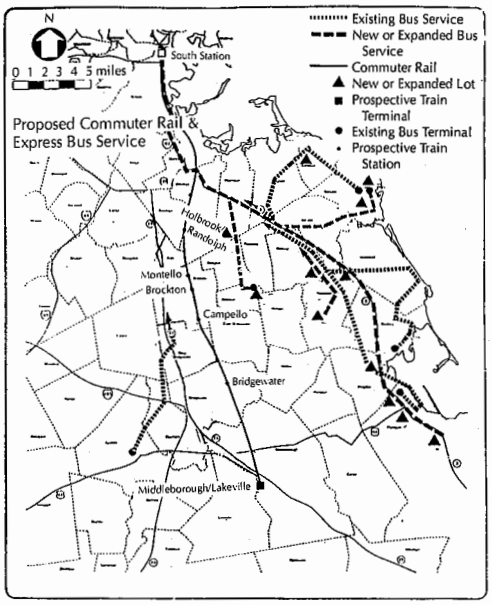


FIGURE 4. Alternatives evaluated in detail in the DEIS/DEIR.

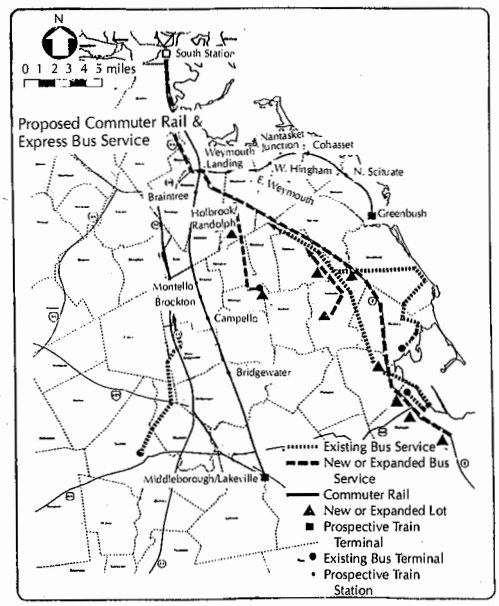
TSM Alternative



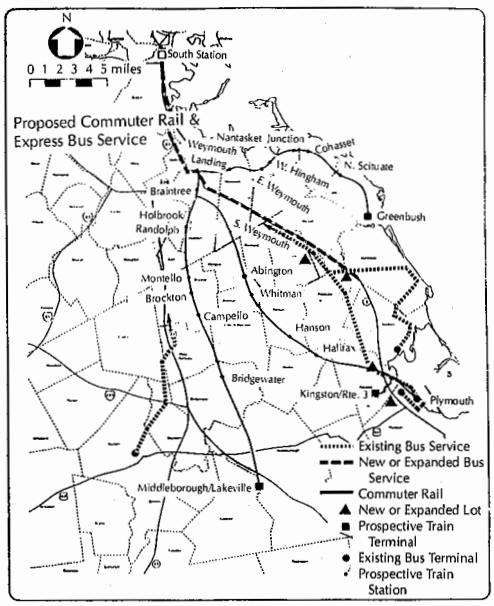
Alternative 3a



Alternative 3c



Alternative 3d



Courtesy MBTA

and Plymouth Lines. Alternative 3c included commuter rail service on the Middleborough and Greenbush Lines. Alternative 3d restored commuter rail service on all three lines. In all four alternatives, TSM improvements would be included in the Old Colony area corridors not serviced by commuter rail.

Detailed analyses were conducted on these six remaining alternatives following UMTA's guidelines for transit project planning. These

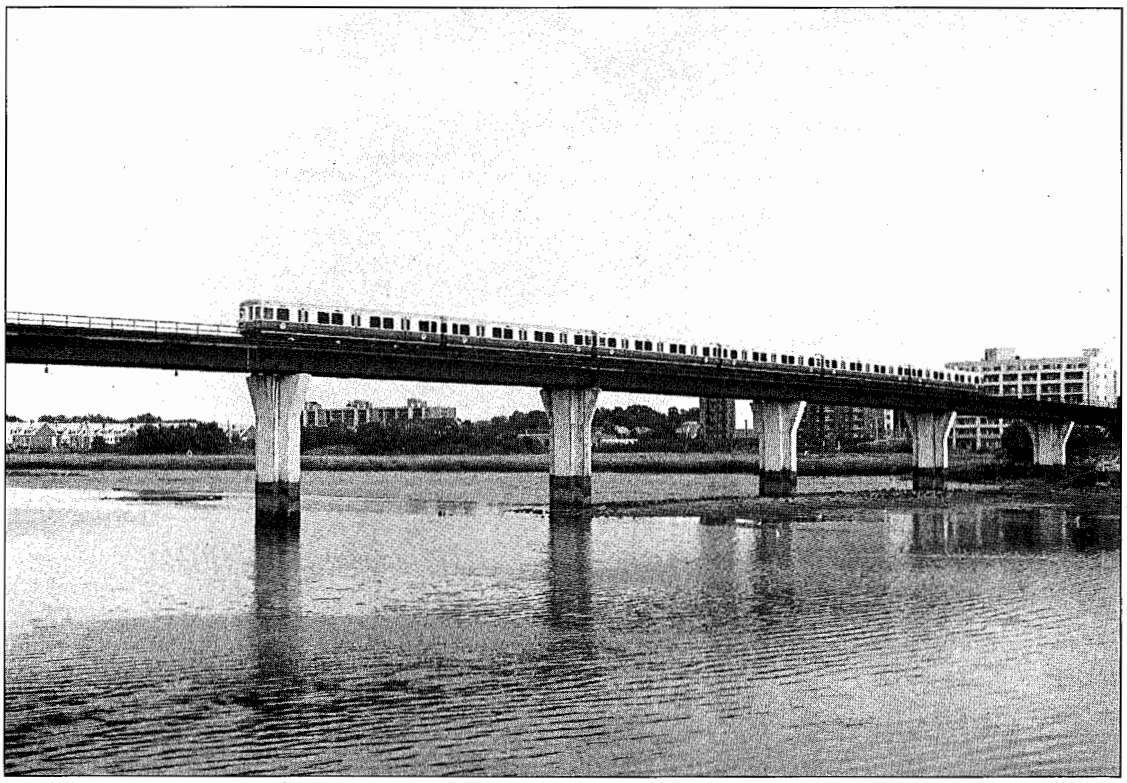


FIGURE 5. A view of the Neponset River. The Old Colony commuter rail bridge will parallel the Red Line transit bridge (shown) and will provide the same clearance above water.

analyses covered ridership projections and other transportation impacts using a ridership forecasting model that was accepted by UMTA for the Old Colony study area; capital costs based on conceptual (ten to 30 percent) engineering plans; operating and maintenance cost estimates based on detailed operating plans and UMTA's cost build-up approach; environmental concerns covering social and economic impacts, traffic and parking impacts, impacts to the natural environment, impacts on cultural resources and impacts during construction.

The financial capacity of the MBTA and the state to undertake the most expensive of the six alternatives, Alternative 3d, was evaluated. The alternatives were compared with respect to their cost-effectiveness in achieving the project's goals and objectives and their fairness in allotting the benefits and costs/impacts across different population groups. The trade-offs among the alternatives in terms of their costs, impacts and ability to achieve the project's goals and objectives were deliberated.

Alternatives Being Advanced in the FEIS/FEIR. The DEIS/DEIR was completed and circulated in the May 1990. Three public hearings were held, many other public meetings were conducted and almost 2,000 public comments on the DEIS/DEIR were received.

Because the information presented in the DEIS/DEIR and comments received on the DEIS/DEIR indicated that the response to comments and the resolution of environmental issues related to the Greenbush corridor could not be addressed in the same timeframe as those related to the other Old Colony corridors, the MBTA and UMTA decided to proceed with the completion of the FEIS/FEIR for transit improvements in the three other corridors.

References to the Greenbush Line commuter rail improvements that are an integral part of Alternatives 3c and 3d, the Greenbush corridor express bus and related park-ride services that

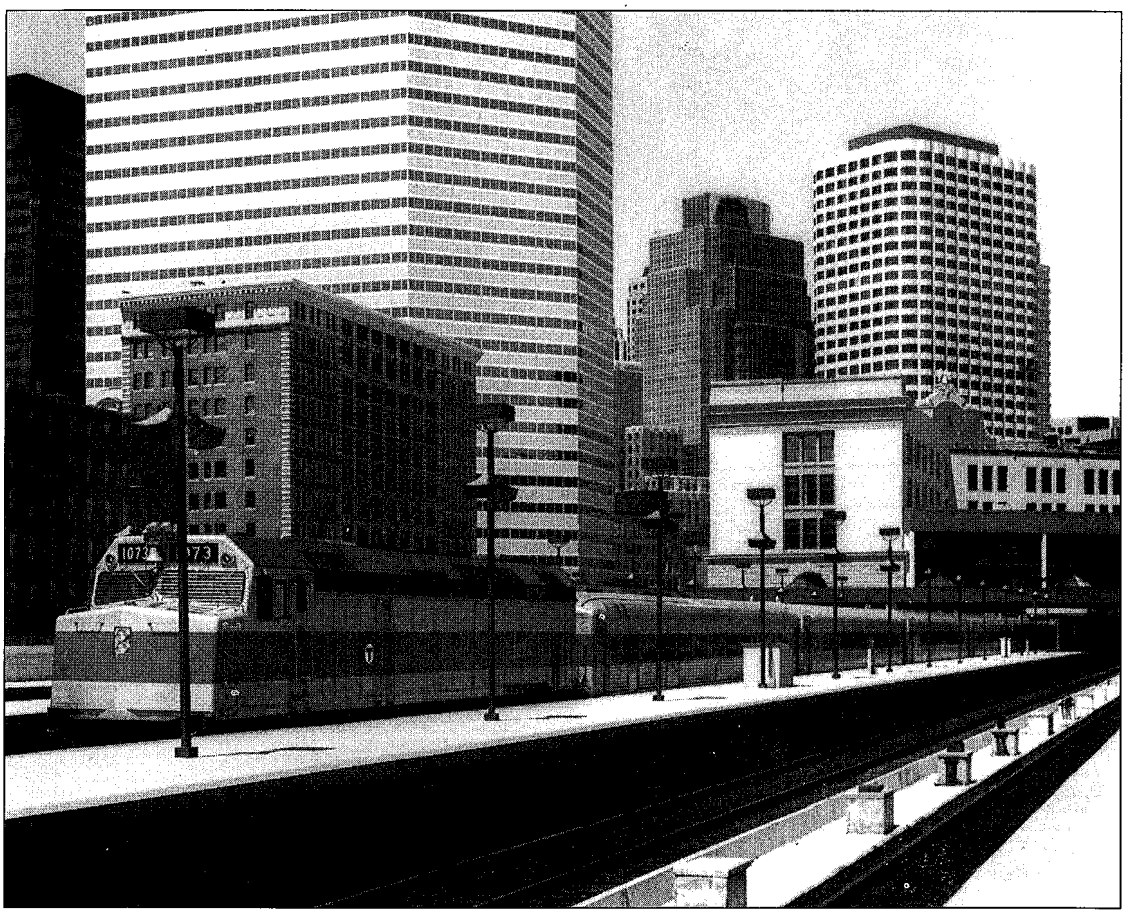


FIGURE 6. A view of South Station in Boston. Old Colony trains will use the recently renovated station. Operational analyses for the project included occupancy simulation of South Station tracks by Old Colony trains, other commuter rail trains and intercity trains.

are included as elements of Alternatives 3a and 3b, and the service area of the Greenbush corridor, will not be included in the evaluation chapters of the FEIS/FEIR. Transit improvements on the Greenbush corridor will be assessed and documented in a supplemental environmental document.

The Old Colony Project has created many interesting and challenging design and operations planning issues that needed to be resolved and that have had to be continuously better defined as the project has developed. The most prominent of these problems are:

- The design of the 1,200-foot-long, two-track Neponset River Bridge over a major navigable urban waterway in an area immediately adjacent to existing crossing bridges, with extremely limited on-shore work space and with potentially contaminated dredge spoils to be handled (see Figure 5).
- The development of operational plans for a railroad system that will provide market-driven schedules on what is essentially a one-track system with active freight operations. The effort has involved implementing simulation approaches in order to optimize train operations, equipment usage and passing siding locations that will attain 20 to 30 minutes peak period headways (see Figure 6).
- The location and design of 21 train stations, three train layover facilities and 15 park/ride lots consistent with the operational requirements of the project, a min-

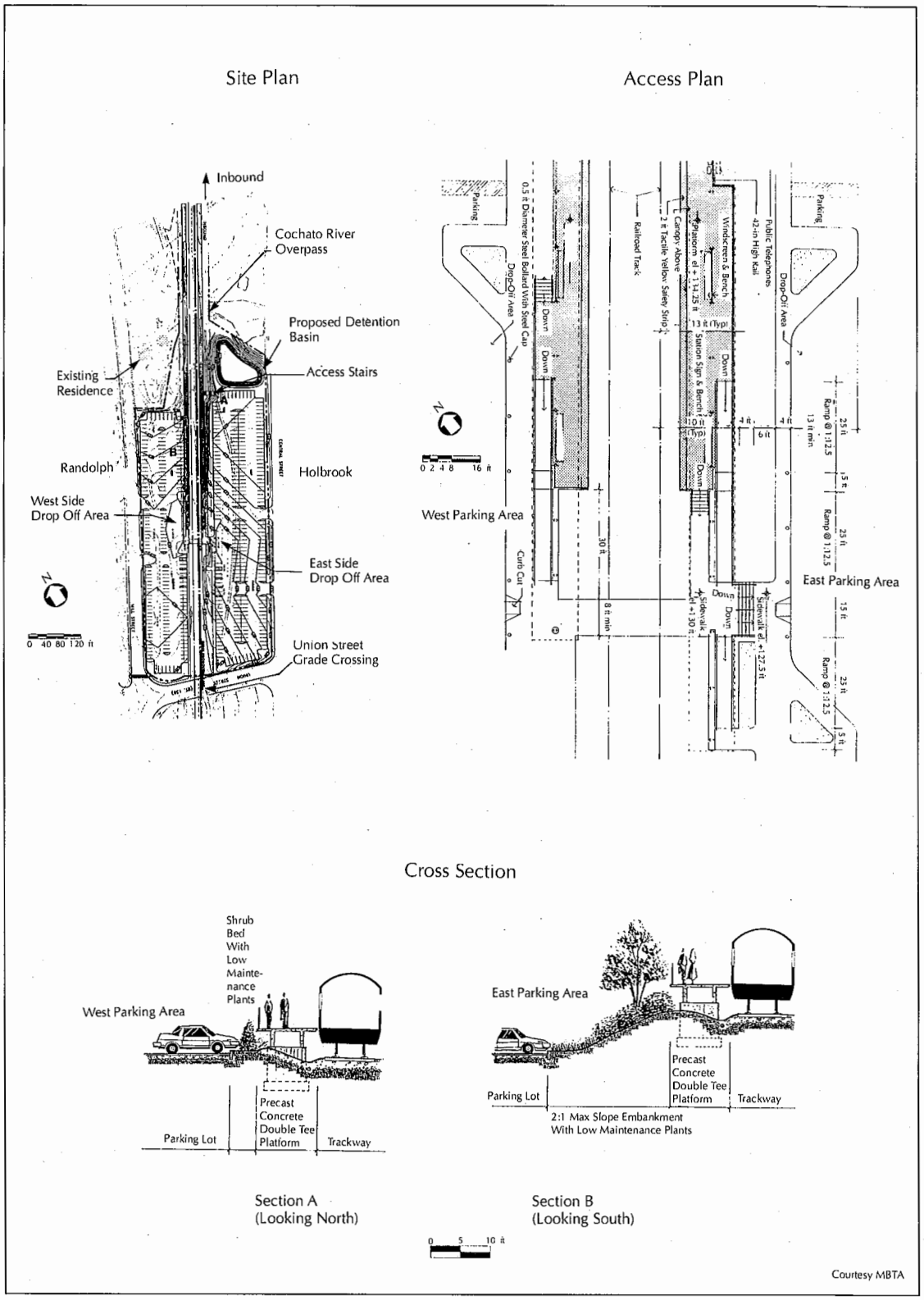


FIGURE 7. The Holbrook/Randolph station.

FIGURE 8. Old Colony stations will be similar in design to recently built MBTA commuter rail stations such as the Forge Park/495 station (shown), but with full-length, high-level platforms.

imization of environmental impacts and a sensitivity to the needs of the individual communities. Each of these facilities represents a major site development project in itself. The proposed Holbrook/Randolph Station, for example, is shown in Figure 7.

- The design of station platforms and access elements in order to fully comply with the Americans with Disabilities Act of 1990 and state ordinances regarding accessibility to public transportation. The system is being designed with full-length high-level platforms at every station (see Figures 7 and 8). Providing a cost-effective approach to these major design elements is an essential part of continued design development.

The Evaluation of Transportation Alternatives

Table 3 contains some of the results of the evaluation done to date on the four alternatives that are discussed in the FEIS/FEIR. The No-Build Alternative constitutes the base line against which all the other alternatives were evaluated. This alternative includes existing and committed transportation facilities. Clearly, it would have the lowest cost among the alternatives considered, but it would not provide additional travel options nor would it improve access to and from Boston and the Old Colony area. The No-Build Alternative would not result in a more equitable distribution of transportation benefits and would not support the economic development of Boston and the Old Colony

TABLE 3
Summary of Selected Results by Alternative

Impacts	1 No-Build	2 TSM	3a Middleborough (Modified)	3b Middleborough Plymouth (Modified)
Capital Costs (Million)*	$ 0.0	$49.1	$281.0	$453.0
Operating & Maintenance Costs (Million)**	$12.3	$16.5	$ 20.2	$ 23.8
Transportation Impacts				
Annual Transit Passenger Miles (Million)	176	189	215	237
Average Morning Peak Period Project Area Transit Travel Time (Minutes)***	53	51	46	45
Percent of Old Colony Residents Within 40 Minutes of Transit Travel Time to Downtown Boston	0	4	33	40
Annual Transit Passenger Miles on Dedicated Rights-of-Way (Million)§	58	58	110	144
Morning Peak Period Transit Inbound Ridership	12,700	13,400	15,700	17,400
Transit Mode Share for Morning Peak Period Inbound Trips Between Study Area & Boston & Cambridge (Percent)	17	18	21	24
Congestion Time Reduction on the Southeast Expressway§§ (Minutes Each Direction)	—	6	30	49
Reduced Parking Demand as Percent of Current Capacity				
From Boston PMA	—	1	4	6
From Red Line South Shore Parking	—	0	0	2
UMTA New Trip Cost Effectiveness Index§§§	—	$16.49	$15.70	$15.95

* Escalated to mid-point of construction.
** Annual costs in 1989 dollars.
*** Average of access, wait & line-haul time to Boston (South Station) for all current Old Colony riders in all transit modes.
§ Includes commuter boat's line haul miles.
§§ North of Southampton Street, Boston. Congestion time reductions assume no change in drivers' behavior regarding time & route of travel.
§§§ Added annualized costs (capital & operating costs, net of travel time savings) per added new transit rider, build alternatives relative to no-build.

study area.

The three build alternatives all contribute in some degree to achieving the project goals. The TSM, Alternative 3a and Alternative 3b are progressively more successful in meeting the project goals, but at an increasing cost and an increasing extent of physical impact.

When comparing all the build alternatives for the Old Colony study area, the TSM would result in the lowest capital and operating outlays. The TSM would not address deficiencies in the transportation system at the regional and corridor levels. While the TSM may increase travel options somewhat, it would do little to reduce highway congestion and commuting time, and would be less attractive than commuter rail in drawing new transit riders. Consequently, the TSM alternative would be less effective in achieving the goals and objectives of the Old Colony study area. The commuter rail alternative would divert between 3,000 to 5,000 morning peak period auto commuters to transit, resulting in reduced highway congestion and parking demand in downtown Boston (up to six percent of the current capacity). Referring to Table 3, the TSM alternative would be less cost-effective than the two rail alternatives.

Of the two rail alternatives, modified Alternative 3a would require fewer capital and operating dollars. It only involves the Main and Middleborough Lines. The addition of the Plymouth Line under modified Alternative 3b provides major incremental improvements to the effectiveness of the service provided by the Middleborough Line alone. The addition of the Plymouth Line allows the high capital cost of the Main Line from Braintree to Boston to be distributed over a significantly higher level of use, but because of the added capital and operating costs, the cost effectiveness (added cost per added transit rider) of the two alternatives is about the same.

Modified Alternative 3b would be most effective in supporting the stated transportation and development objectives and goals of the project. This alternative would support the regional development and transportation strategy and address deficiencies in the current transportation system at the regional and corridor levels. Compared with the No-Build, TSM and modified 3a alternatives, the modified Alternative 3b would provide the most travel options, attract the most transit riders, maximize transit travel time reductions, reduce highway and existing transit facilities congestion, most effectively reduce the regional isolation from Boston and result in a more equitable distribution of transportation benefits. Based on the MBTA's commitment to improved service, this alternative is being advanced as the preferred alternative in the FEIS/FEIR.

Next Steps to Project Development & Conclusions

The MBTA is close to completing the FEIS/FEIR. Final design and construction will begin by the end of 1991 once the FEIS/FEIR is circulated and UMTA announces its Record of Decision. The Greenbush Line Section 4(f) Evaluation is expected to be completed shortly thereafter, at which time decisions about the type of transportation improvements to be undertaken in that area will be made.

The commitment and effort of the MBTA and UMTA to improve transportation services in Southeastern Massachusetts, coupled with sound planning and engineering practices and a thorough public involvement program, have been key ingredients in moving forward one of the major commuter rail restoration projects undertaken in recent history. The substantial involvement of the public in the project from the earliest stages of the scoping and screening process and throughout the environmental review has built a consensus around the project's new ideas and proposals, facilitated the decision making in many design issues, and minimized problems early in the planning and project development. Most importantly, thanks to this timely public involvement, the project has been directed on a course of development that not only meets the transportation needs of the region and is technically and environmentally sound, but also reflects the perspective of the communities and people it will serve.

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DOMENIC E. D'ERAMO *received a B.S. in engineering from Brown University in 1958. He is a Vice President of Sverdrup Corporation and Manager of the firm's New England Division. With Sverdrup for nearly 30 years, he has concentrated on managing the design and construction of major transportation facilities. He has played key roles in several highly visible local projects ranging from the MBTA's Red Line Northwest Extension and the design and construction of the ACEC award-winning I-391 from Chicopee to Holyoke, Massachusetts, to ongoing projects such as Boston's Third Harbor Tunnel, the Old Colony Railroad Rehabilitation, and the Sudbury Aqueduct and Connecting Tunnels Rehabilitation. A past president of the Boston Society of Civil Engineers, he is also a Trustee and Treasurer of the Engineering Center Education Trust, and past president of the Engineering Center.*

RODOLFO MARTÍNEZ *holds a B.S. in economics and he has also received an M.S. in transportation, in 1984, and an M.S. in management, in 1986, from the Massachusetts Institute of Technology. Currently, he is a Senior Planner at Sverdrup Corporation, specializing in economic, financial and operations analyses for transportation and capital facility projects. Projects of note in which he has played a major role include the Old Colony Railroad Rehabilitation along Boston's South Shore and the Los Angeles/San Diego Bullet Train Project in California. Prior to joining Sverdrup in 1987, he served as a financial analyst for the World Bank and was a Professor of Economics at the Central American University in El Salvador. He is a member of the American Planning Association, an associate member of the Institute of Transportation Engineers and an affiliate member of the American Society of Transportation and Logistics.*

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Close-In Construction Blasting: Impacts & Mitigation Measures

Simple, practical techniques can be used to minimize potential damage from close-in blasting by minimizing the effects of elastic ground vibrations & non-elastic ground deformations.

ANDREW F. MCKOWN

As development pushes further and further into areas of shallow bedrock, and as existing facilities expand at rock sites, there is an increasing need to perform drill and blast rock excavation in close proximity to existing structures. If not carefully controlled, the release of high energy from the explosives required for the blasting process can have undesirable side effects that could pose a threat to the adjacent structures.

Possible Damage Mechanisms

Some of the undesirable side effects of blasting that could result in damage to nearby structures include:

- Elastic ground vibrations
- Non-elastic (permanent) ground deformations
- Flyrock
- Airblast overpressure

Flyrock can be very dangerous, since it causes real damage and can result in physical injury or even death. Airblast overpressure is generally only a nuisance, since for most blasting where explosives are confined in boreholes by stemming, it would not cause damage to structures. Elastic ground vibrations and non-elastic ground deformations, which are separate and distinctly different categories of ground motion, are discussed here.

The first, and most familiar (most generally understood), side effect from close-in blasting is ground vibration — the elastic ground motions resulting from vibration waves emanating from the detonation source. The second effect is potential permanent ground deformations that result from cratering effects very close to the blast source and from block movement caused by explosive gasses escaping into open joints and seams in the rock.

Elastic Ground Vibrations. When a buried explosive charge is detonated, ground vibrations are generated. Several types of waves make up these vibrations (compression waves, shear waves, Rayleigh waves, *etc.*). Each of these wave types has its own propagation velocity

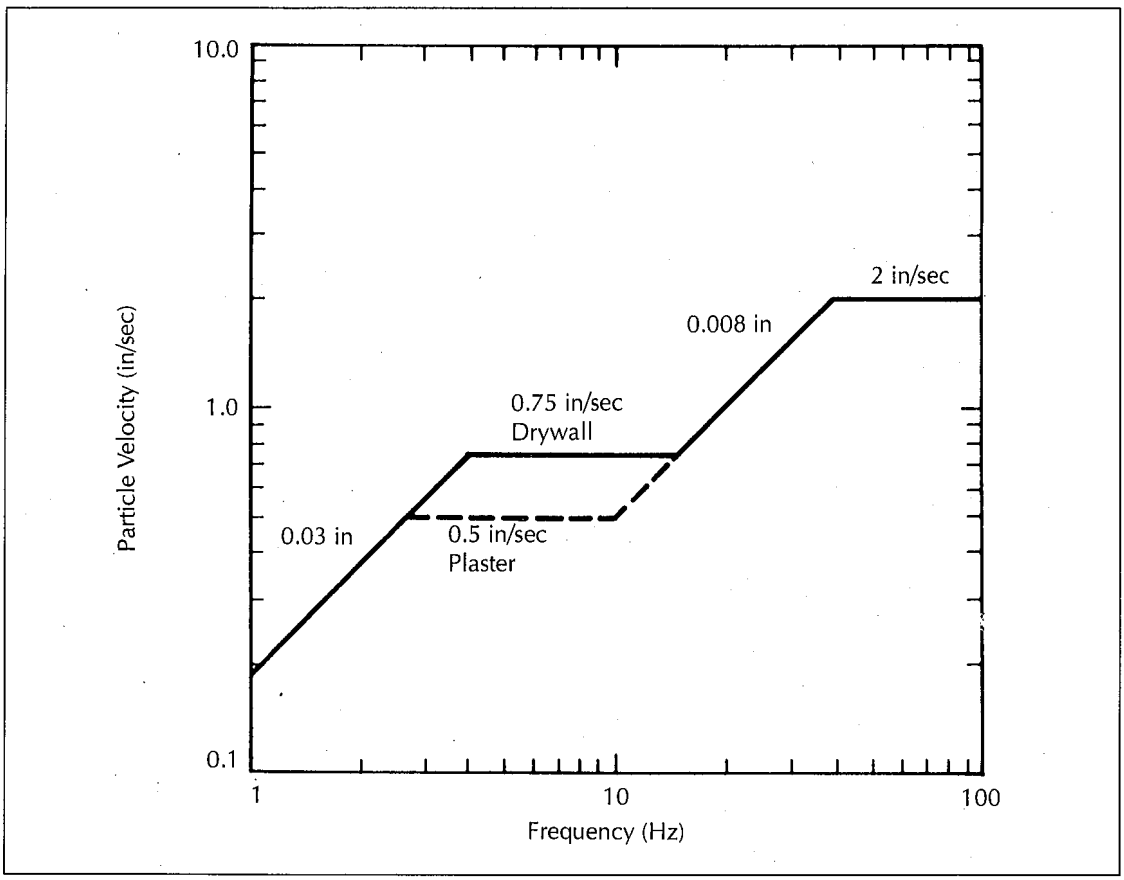

FIGURE 1. Bureau of Mines "safe limits" of blast vibrations for residential structures.

and direction of particle motion.

To simplify an understanding of wave motion, a near-surface detonation may be compared to a stone thrown into a pond. Surface waves travel in concentric circles out from the impact point. A small boat would bob up and down as the waves travel by. In a similar manner, a structure or pipeline would move as families of ground waves travel by. The motion would actually be in three dimensions, although the surface waves (usually Rayleigh waves) generally have the largest particle motion. The ground displacement, or the magnitude of particle motion as the waves travel by, can be measured. For most blasting situations, the ground displacement falls in the range of a few hundredths or thousandths of an inch. The peak particle velocity (PPV), or the speed at which the ground moves up and down (or sideways) as the wave travels by, can also be measured. PPV is measured in inches per second (in/sec), and differs from the wave propagation velocity, or speed at which the wave travels through the ground. The vibration frequency is the number of waves passing a given point per unit time, and is measured in Hertz (Hz), or cycles per second.

PPV, displacement, D, and frequency, f, can be related for a sinusoidal wavetrain as follows:

$$D = PPV/2\pi f$$

Although blasting vibrations are not generally sinusoidal, the approximation is generally reasonable for estimating displacement.

PPV is the ground motion parameter generally used to assess the damage potential of blast vibrations. Many states have regulations that limit the maximum PPV of ground vibrations from blasting to two inches per second at adjacent structures. This criterion was developed based on research of blast damage to residential

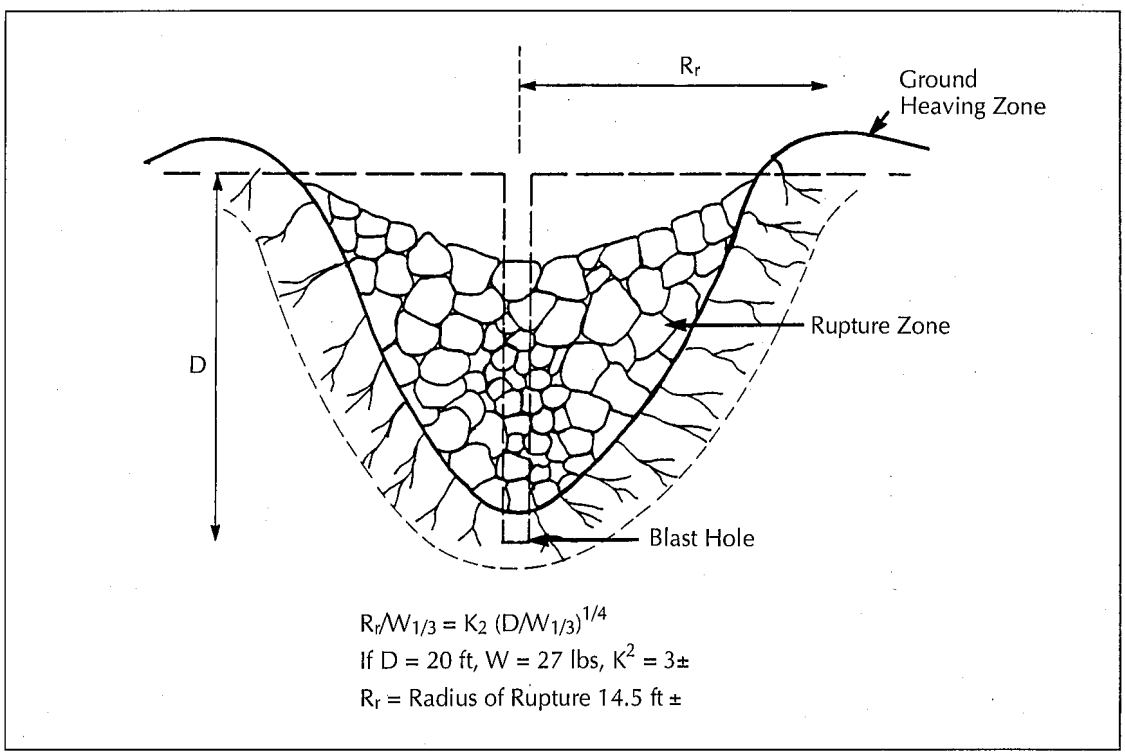

FIGURE 2. Cratering rupture zone from the detonation of a confined explosive charge.

structures conducted by the United States Bureau of Mines.[1] More recent work by the Bureau has indicated that these limits, in some cases, should be reduced.[2] Based on the vibration frequency and type of interior wall construction, it recommends limits ranging from 0.5 to 2 inches per second, as indicated in Figure 1.

The lower portion of these PPV criteria are based primarily on surface mine blasting (most frequently using large charge weights in large diameter holes) at great distances (generally, a thousand feet or more) from residential structures. The resulting ground vibrations at residential structures sometimes occur at the relatively low frequencies that are in the range of natural frequency of structures. Typical whole-structure natural frequencies of residences fall in the range of about five to 15 Hz, and vibrations in those frequencies may be amplified more than the higher frequencies. The criteria are intended to minimize the possibility of cosmetic damage to residential structures, such as threshold cracking of plaster walls or wall board, or minor extension of existing cracks.

The situation in close-in construction blasting is generally quite different from quarry or mine blasting. In this case, blasting is very close (say, less than 50 to 100 feet) to structures, with small charge weights in small diameter holes. This type of blasting most often results in ground vibrations with very high vibration frequencies, little to no amplification, and small ground and structure displacements.

The adjacent structure is often not a house with plaster walls. Instead, it may be a reinforced concrete or steel frame commercial or industrial building, a bridge abutment, a pipeline, tunnel or other buried structure, or other engineered structure that is much more resistant to damage than a residential structure. PPVs have been observed in excess of five in/sec at steel frame, masonry and reinforced concrete structures and over six in/sec at gas transmission pipelines with no damage. Oriard[3] and the Southwest Research Institute[4] have reported peak particle velocities of over 20 in/sec at above ground structures and up to about 50 in/sec at underground pipelines without damage.

Despite these differences in vibration char-

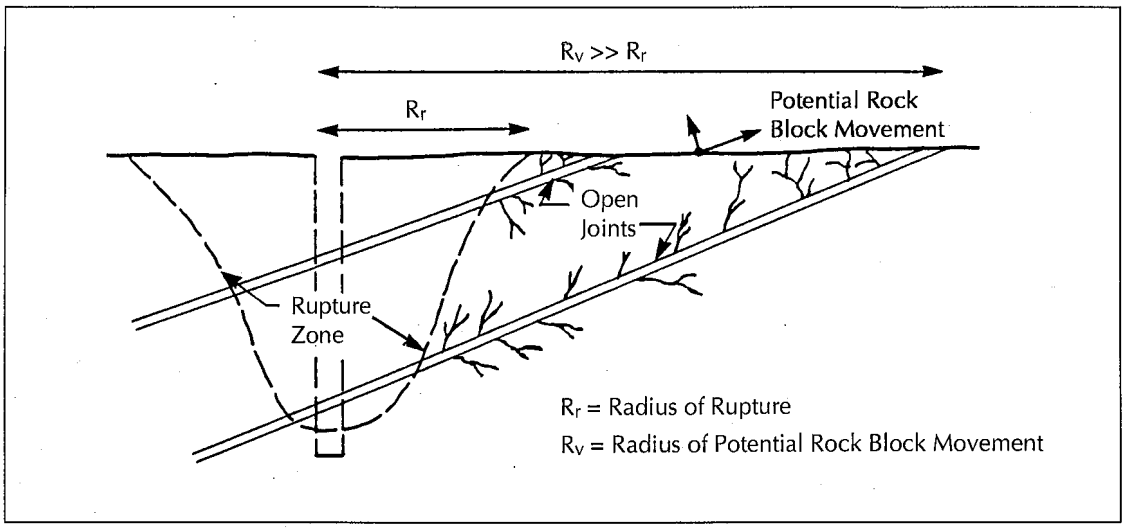

FIGURE 3. Potential rock block movement from gas venting during blasting.

acter (low frequency, high displacement *vs.* high frequency, low displacement) and damage resistance of structures, the two in/sec "safe limit" for residential type structures continues to be applied when blasting close-in to these more massive structures. This method results in use of very expensive non-blasting fragmentation techniques in some cases, longer construction times in others, and generally adds unnecessarily to the construction cost.

Permanent Ground Deformation. If a blast has a high degree of confinement, a cratering rupture can occur such as shown in Figure 2 that can cause permanent, or non-elastic, ground deformation. In addition, explosive gasses can enter open joints and heave the ground or move rock blocks at larger distances from the detonation point (see Figure 3). Dowding cites a case where a 50- by 30- by 30-foot high block of rock was moved 2.5 inches by an eight-hole (approximately 150 pounds total weight) blast round detonated nearby with large burden.[5]

In close-in blasting, these permanent ground deformations are often more of a threat to above ground structures and underground pipelines than the generally better understood and more feared ground vibrations. For example, Barenberg has correlated pipeline damage to permanent ground displacement caused by the 1971 San Fernando Earthquake.[6] These data indicate that for twelve-inch diameter or larger water main distribution pipelines, made of cast iron and steel, there was no damage (less than one break per kilometer) at ground displacements up to 1.2 inches. At two inches of displacement, there were about four breaks per kilometer, while at three inches there were about ten breaks per kilometer. In order to get one inch of ground deformation from elastic ground vibrations, even at a low blast-induced frequency of ten Hz, the PPV would have to be over 60 in/sec.

Potential Situations for Blast-Induced Permanent Ground Deformations

There are several factors that may contribute to high blast-induced ground deformations. Although not intended as an all inclusive listing, some factors and situations that could lead to such problems are described below:

1. *High confinement.* One of the major factors leading to large ground deformations, both elastic and non-elastic, is high confinement. This condition may result from too large a burden (distance to a free face), low angles of breakage (primarily less than 90 degrees), absence of a free face (or only one free face) or other situation that tends to restrict the fragmentation and movement of rock.

Some types of blasting, such as that for shafts and tunnels, which have only one free face and low angles of breakage, have a tendency to generate higher ground deforma-

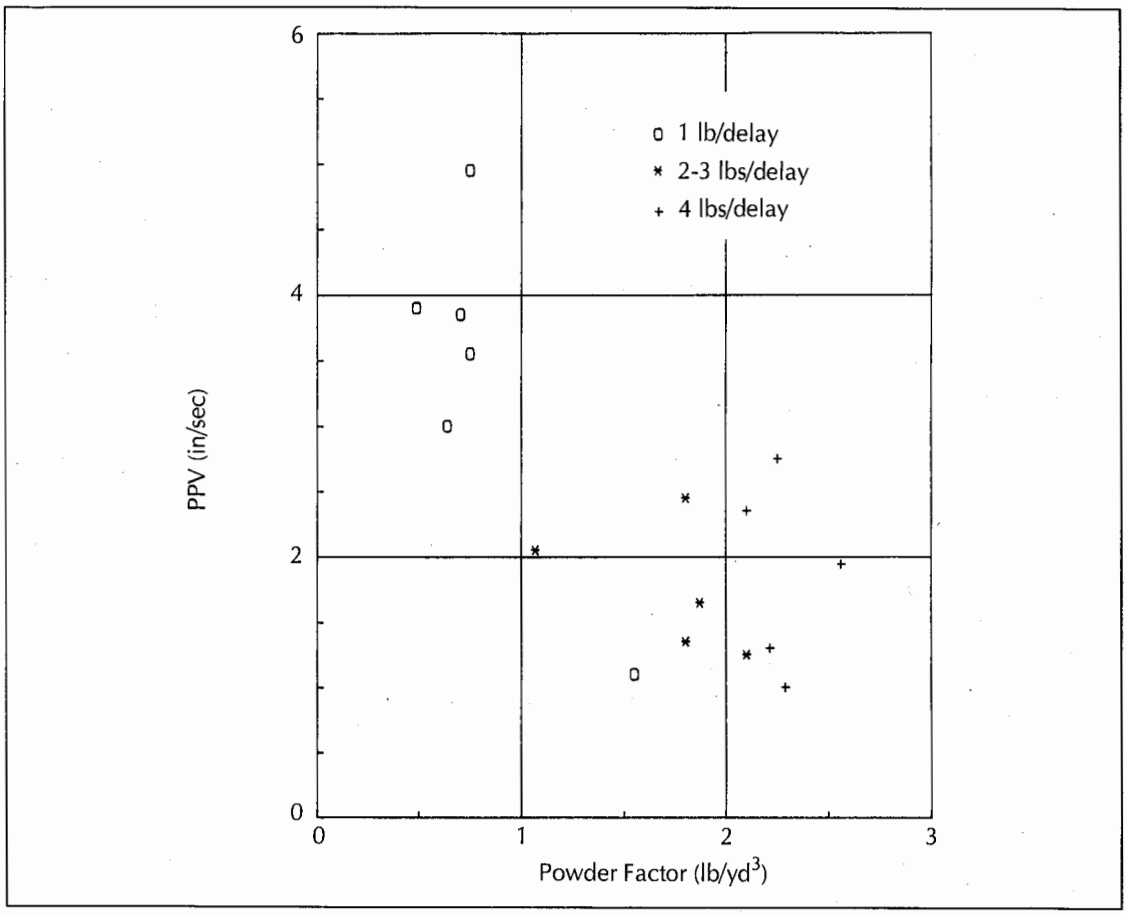

FIGURE 4. Powder factor *vs.* PPV for a trench blasting project near bridge abutments.

tions. Similarly, pre-splitting, which by its nature has complete confinement, can lead to relatively higher ground deformations. In open cut excavations, interior corners often have low angles of breakage.

2. *Lack of confinement.* As opposed to the high confinement associated with interior corners, exterior corners can be a problem for the opposite reason. Because of the two vertical free faces, there is a tendency for joints to open and for blocks to move due to a lack of confinement at one face when detonating charges at the other.

3. *Open joints and seams.* These geologic conditions, and other structural discontinuities in the bedrock mass, can be avenues for the escape of explosive gasses, which can lead to high non-elastic deformations through rock block movement.

4. *Concentrated loading.* Heavily concentrated charges, if they are near the perimeter of excavations, can result in overbreak and rock block movement outside the limits of excavation.

5. *Low powder factor.* If the powder factor (weight of explosive per unit volume of rock) is too low for optimum fragmentation of the rock, the explosive energy will be utilized in undesirable side effects such as ground heaving, block movement or higher levels of elastic vibrations.

As an example, Figure 4 provides a plot of powder factor *vs.* PPV for a trench blasting project that was in close proximity to bridge abutments and wingwalls.[7] As the blasting contractor approached the bridge, where blasting was required within several feet, the contractor systematically reduced the maximum charge weight per delay to about one lb/delay

by reducing the hole loading, without reducing the hole spacing or burden. The result was incomplete fragmentation, high bottom (intact rock remaining above design subgrade level) which had to be reshot, and higher levels of ground vibrations that actually increased as the charge weight per delay was decreased. When hole spacing and burden were finally decreased, which increased the powder factor, the contractor found that the charge weight per delay could be increased up to as much as four lb/delay while still maintaining lower PPVs at similar distances, because the explosive energy was then going into fragmentation, rather than ground vibration.

Protection Approaches

There are several steps that can be taken in order to minimize the potential for high elastic or non-elastic ground deformations at structures when blasting nearby. First, the work is made easier and the potential for damage is minimized, if blasting starts farther from the structure and moves towards the structure. As the blasting approaches the structure, similar improvements occur if the rock is removed from the blast area after each round, in order to provide more relief for the following round. Observations should be made after each round for cratering effects or evidence of rock block movement due to the venting of explosive gasses.

It is helpful if drillers keep detailed, written records that identify the depths of the seams and fracture zones that are encountered. It is also helpful to study the exposed vertical rock face after each round so that open joints that could cause gas venting can be identified. If open continuous joints are found at locations and orientations that could result in rock block movement adjacent to or below the structure, grouted rock dowels may be used to reinforce the rock between the blast area and the structure. In addition, the blasting contractor should take precautions, such as stemming through open joints (filling the hole with course sand to fine gravel sized material), to avoid placing explosives in areas where gas venting could occur. Line drilling (unloaded perimeter holes), at a spacing of two to three times the hole diameter, can be useful in venting explosive gasses that might cause block movement or heaving outside the limits of excavation.

Smaller blast rounds may be desirable as the blasting approaches the structure in order to minimize the explosive charge weights and mitigate impacts in the event the round does not produce the expected results. In addition, carefully executed perimeter control procedures (generally, line drilling or cushion blasting) should be utilized at the excavation limits if a structure is close by, in order to minimize potential damage to rock supporting the structure or rock block movement under the structure. Typically, perimeter control techniques involve closely spaced, lightly loaded (or unloaded in the case of line drilling) holes at the perimeter of a rock excavation in order to minimize overbreak and damage to the remaining rock. In pre-splitting or pre-shearing, the perimeter holes are lightly loaded and detonated before the adjacent production blasting is performed, in order to create a narrow fracture zone between holes to which the subsequent production holes can break. With cushion blasting, or trim blasting, perimeter holes are fired after the main excavation has been blasted in order to remove the final berm of rock.

Several additional steps may be taken in order to minimize potential ground vibration impacts on the structure. First, it is desirable to specify safe but realistic PPV limits in the contract documents based on the type, age, condition and use of the structure, as well as the proximity of required blasting to the structure. When blasting close-in to structures bearing on rock, vibration frequencies are likely to be over 100 Hz, and are often much higher. At these high frequencies, amplification is unlikely and displacements are low. For example, a PPV of four in/sec at 100 Hz would result in an elastic displacement of only 0.006 inches. Blast vibration monitoring should be conducted to check for conformance with specified limits.

Blasting contractors should submit details of proposed blasting procedures for review by those experienced with close-in blasting procedures, preferably by the person who prepared the specifications. It is also desirable for that person to make periodic site visits in order to observe the blasting procedures and results, and to work with the blasting contractor in the event that rock conditions or blasting results differ

from those that were anticipated.

Case Histories

Three case histories are presented that detail experience with blasting close-in to various types of structures. The first case history is that of an office park development that required a rock cut adjacent to a gas transmission pipeline. The second involved blasting adjacent to an existing hospital. The final case history involved blasting for an underground library addition on a college campus, immediately adjacent to the existing library and adjacent to and below an existing office/classroom building.

Office Park Development. Drill and blast rock excavation was required for a hotel/office/garage complex near a 24-inch diameter gas transmission pipeline on Route 1 in Saugus, Massachusetts. At one end of the site a 25-foot rock cut was required immediately adjacent to the pipeline right-of-way as shown on Figure 5. Because of the close proximity of the blasting to the pipeline, and because the granitic bedrock had open, near horizontal joints, there was concern about possible ground heaving or block movement under or adjacent to the pipeline. In order to minimize the possibility of rock block movement from blasting, several steps were taken:

1. The blasting contractor was required to submit a report on intended blasting procedures for review prior to starting work.
2. The blasting contractor was required, prior to any blasting within 100 feet of the pipeline, to install near vertical grouted dowels immediately behind the perimeter of the excavation. The purpose of this rock support procedure was to first fill any open joints with grout so explosive gasses would not have an escape route and, second, to reinforce the rock at the excavation limits in advance of blasting.
3. The blasting contractor was required to start blasting away from the pipeline and cautiously advance towards the pipeline. Within 75 feet of the pipeline, the contractor was required to remove all blasted rock from the face of the excavation after each round, so that adequate relief would be provided for the next round and so that the geology could be observed.

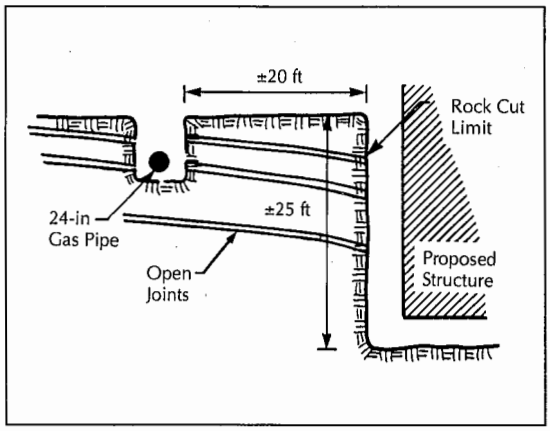

FIGURE 5. Schematic section view showing the proximity of blasting to a gas pipeline and open, near horizontal joints.

4. Careful perimeter control techniques were required at the excavation limits. Presplitting, cushion blasting or line drilling were all allowable methods of perimeter control, and guidelines were provided for hole spacing and loading for each technique.

In addition to these four steps, blast-induced ground vibration limits of five in/sec PPV were specified at the ground surface at the pipeline.

The blasting contractor chose to pre-split the rock at the perimeter and used four- by four-foot spacing for production holes within 30 feet of the excavation limits, with a powder factor of about one pound per cubic yard. Grouted dowel locations and pre-split procedures are shown schematically in Figure 6. The blasting was successfully carried out, with no rock block movements or other detrimental impacts on the pipeline. PPVs at the pipeline exceeded four in/sec without damage.

Hospital Addition. Drill and blast rock excavation was required immediately adjacent to existing hospital structures in two areas at Carney Hospital in Boston. The first area was a foundation excavation for an addition within an existing courtyard. Rock excavation was required within three feet of an existing three-story brick hospital building to a depth about ten feet below the exterior footings, which were bearing on bedrock.

The rock was a moderately to severely weathered conglomerate with an average

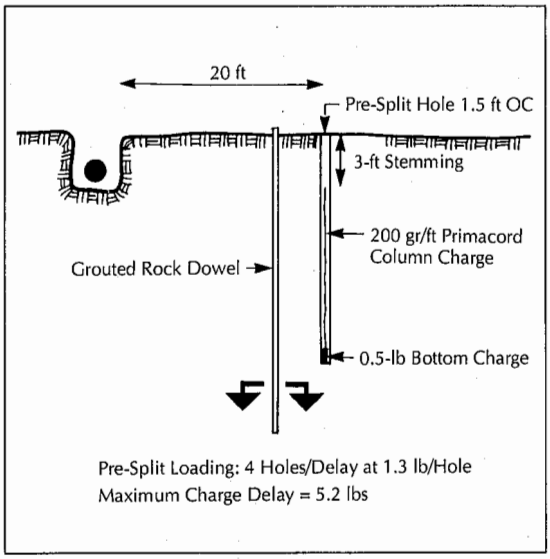

FIGURE 6. Section showing rock dowels, pre-splitting to prevent rock block movement.

rock quality designation (RQD) of about 25 percent. RQD is the percentage of less than severely weathered pieces of rock core within a core run that are greater than four inches long. An RQD above 90 percent denotes an excellent quality rock mass, while an RQD less than 50 percent is indicative of a poor quality rock mass. The rock was heavily jointed, with one joint set dipping into the excavation at an orientation of about 30 to 40 degrees from horizontal.

As a result of the relatively poor quality bedrock below the existing footings, the adverse orientation of joints and the close proximity of required rock excavation, there was concern about overbreak or rock block movement under the existing building footings. Therefore, several requirements to address these conditions were included in the project plans and specifications. The blasting contractor was required to start work in the middle of the infill area and work out towards the buildings, using small rounds and relieving the face after every round. In an area where the excavation wrapped around a corner of the building, near vertical grouted steel dowels were required to be installed prior to blasting within 50 feet. At the limits of the excavation, cushion blasting or line drilling was required.

With regard to elastic ground vibration, a maximum PPV was specified that was based on the distance away from the structure and is presented in Table 1.

TABLE 1
Maximum PPV at the Brick Structure for the Hospital Addition

Distance (ft)	PPV (in/sec)
> 30	1.9
20 to 30	3.0
10 to 20	4.0
< 10	5.0

Close coordination was required between the blasting contractor and hospital personnel so that sensitive equipment or procedures within the affected portion of the building could be temporarily suspended if necessary for several minutes during each blast.

At the limits of the excavation, cushion blasting or trim blasting was used by the blasting contractor to remove a final three- to four-foot wide berm of rock after production blasting was completed. Figure 7 provides a view of the final rock berm left just before cushion blasting. The twelve-foot deep, 2.5-inch diameter cushion blast holes were spaced at one foot on center and loaded with a 0.5-pound bottom charge and a column charge of 400 grains per foot detonating cord (0.06 lb/ft). The holes were fully stemmed around the detonating cord and the upper two to three feet of the hole was left unloaded and stemmed.

The results of the cushion blasting can be seen in Figure 8. There was very little overbreak, no block movement and no need for rock bolts or other temporary rock support procedures. The maximum recorded vibration at the lowest adjacent building slab was 3.2 in/sec, and no damage was observed to the building.

Additional blasting was required at the hospital site for a utility trench between a hospital building and a parking garage. The trench required rock excavation below one small concrete retaining wall and immediately adjacent to and about ten feet below the corner of another larger concrete retaining wall. Figure 9

FIGURE 7. Final berm prepared for cushion blasting for the hospital infill foundation.

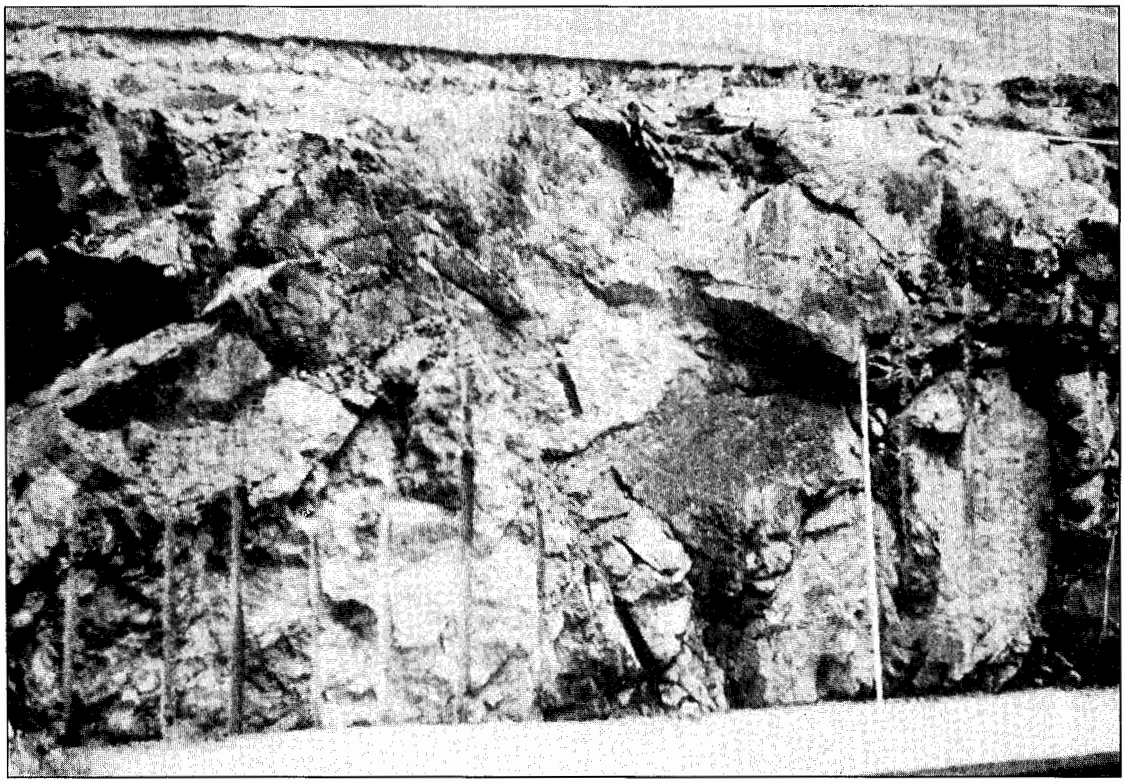

FIGURE 8. Results of cushion blasting for the hospital infill foundation.

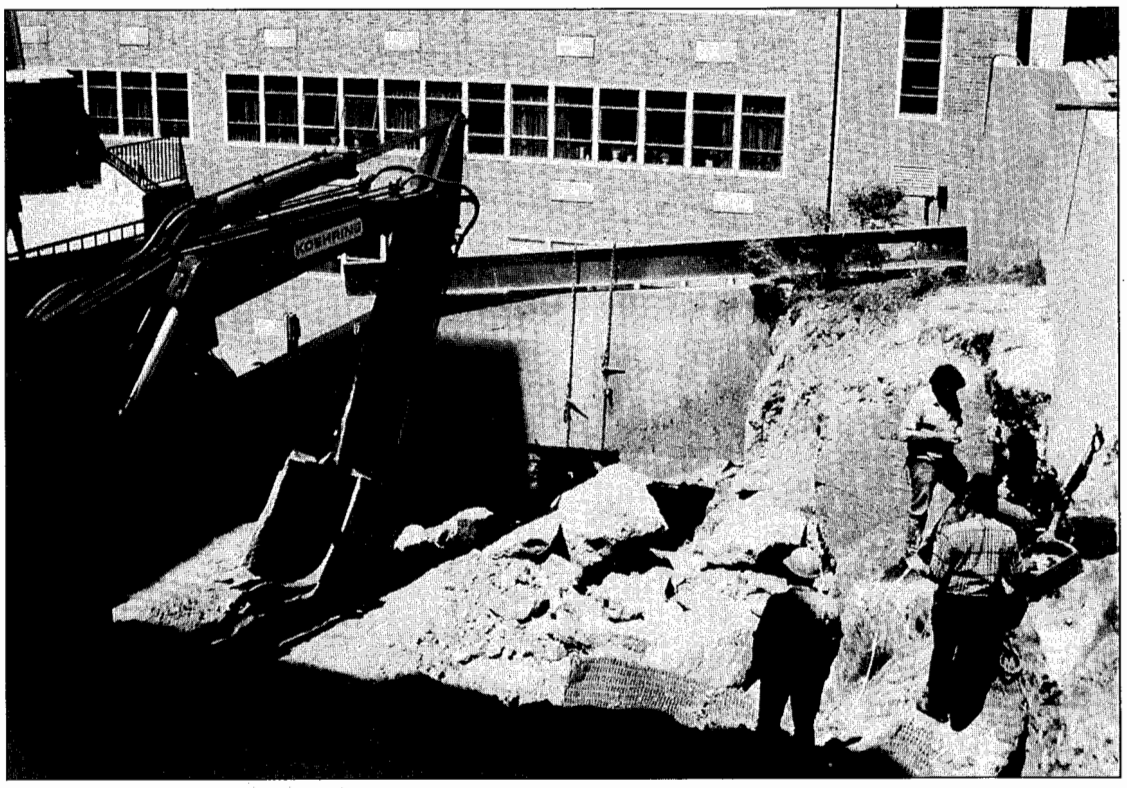

FIGURE 9. A view of the utility trench excavation looking towards the hospital. The small retaining wall supported by needle beam is in the center; the large retaining wall is on the right.

offers a view of the blasting area looking from the parking garage down the trench towards the hospital building. The smaller wall can be seen supported by a needle beam, while the larger wall is to the right. Figure 10 presents another view looking towards the parking garage, with the corner of the larger retaining wall on the left.

Figure 10 also shows a continuous, steeply dipping, moderately weathered joint surface that extends under the corner of the wall. This joint was oriented about 30 degrees from the orientation of the trench perimeter and dipped into the trench excavation at an angle of about 75 degrees from horizontal. There was concern that explosive gasses from the blasting would enter the joint and cause block movement under the corner of the wall. In this area, the blasting contractor left a three- to five-foot wide berm of rock in the area of the wall and used cushion blasting to trim off the remaining rock adjacent to the corner. Cushion blast holes were 2.5 inches in diameter, spaced at two feet on center and alternately loaded with 0.06 and 0.30 lb/ft column loading. The holes were fully stemmed and 2.5-inch diameter unloaded guide holes were drilled midway between each cushion blast hole.

The results of the cushion blasting can be seen in Figure 11. There was no block movement under the wall and no overbreak beyond the cushion blast holes. Vibrations at the wall were estimated to be about eight in/sec with no cracking. At the other retaining wall, vibrations from trench blasting were estimated to be in excess of 20 in/sec with no damage, and at the hospital building measured vibrations were as high as 4.8 in/sec with no observed damage.

Underground Library Addition. Blasting was required for an underground library addition at Cornell University, immediately adjacent to three existing university buildings. Figure 12 offers a view to the southwest toward the two closest structures. On the left is Stimpson Hall, a four-story woodframe structure with stone ma-

FIGURE 10. A view of the utility trench excavation looking towards the parking garage. The larger retaining wall is at the left, with the continuous joint extending under the corner of the wall.

sonry exterior walls and load bearing interior walls. At the time of construction, Stimpson Hall was being used as a laboratory and as office space. To the right of Stimpson Hall is Olin Library, a seven-story reinforced concrete structure. Another three-story masonry building, Goldwin Smith Hall, is located out of sight at the other side of the excavation limits.

The total excavation depth required was about 50 to 60 feet from the original ground surface, through about 25 to 30 feet of soil and 25 to 30 feet of rock. Figure 13 shows the site from the southeast, with Stimpson Hall to the right. Concrete underpinning was used to carry the load of soil supported wall footings down to the top of bedrock. Along the east side of the excavation the 30-foot deep earth excavation was supported with an innovative soil nailing system that consisted of shotcrete sprayed over the earth cut, in combination with grouted steel anchors into the soil mass. A conventional soldier pile and lagging earth support system was used in some areas (as shown in the left of Figure 13). Figure 13 also shows a steel needle beam supporting the center of the exterior wall of Stimpson Hall. A proposed entrance stairwell in that area required blasting for a shaft-like excavation within the building limits.

Bedrock at the site was a gray, horizontally bedded mudstone of the Ithaca Formation. The average intact compressive strength of the rock was about 22,000 pounds per square inch (psi). The overall rock quality as measured by RQD was "good to excellent," with an average RQD of 91 percent. Rock with lower RQD values, which would be classified as "fair" quality rock, was found in several borings near the top of rock where weathering and jointing were more severe.

Figure 13 shows the two primary joint sets

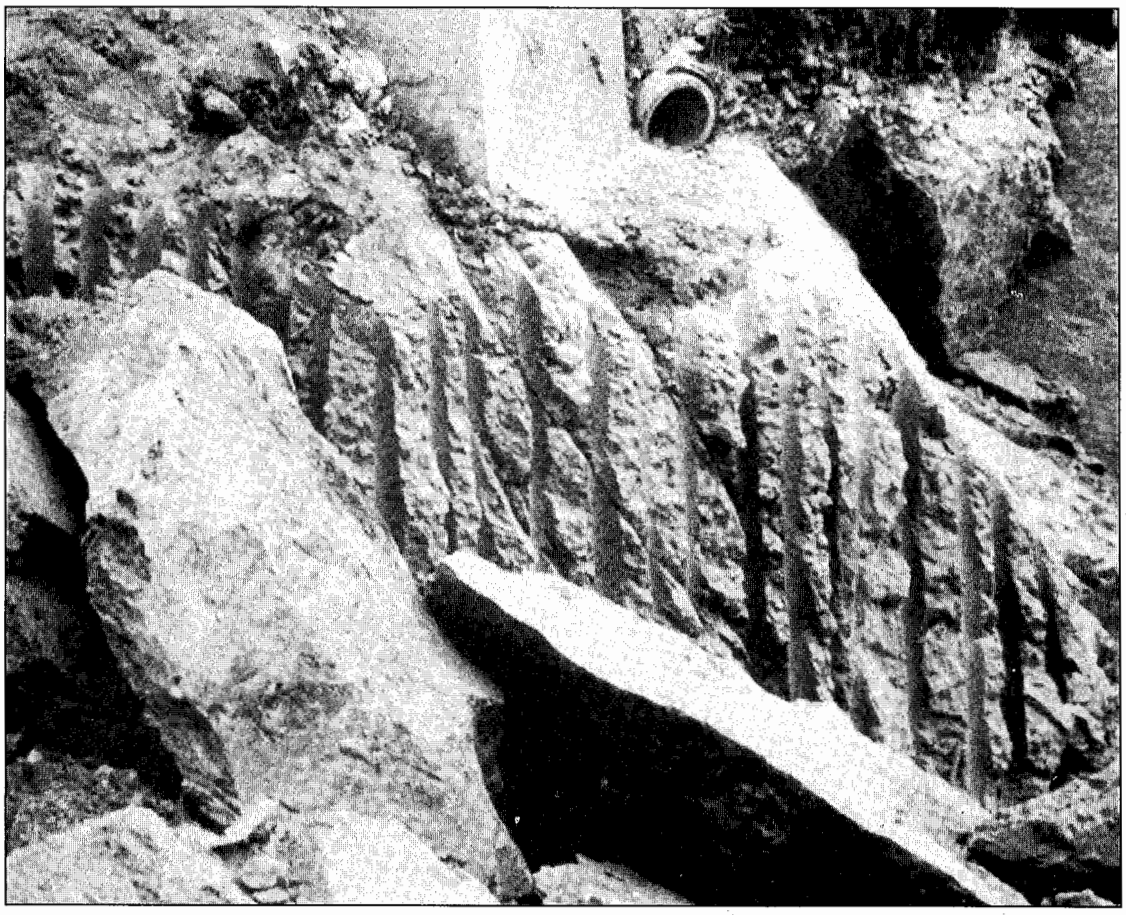

FIGURE 11. Results of cushion blasting at the retaining wall for the hospital utility trench.

that influenced the blasting results (excluding the horizontal bedding plane joints). One joint set, called the "strike joints," was oriented approximately parallel to the face of Stimpson Hall in an east-west direction, with a dip of about 90 degrees (near vertical). The other joint set, called the "dip joints," was oriented perpendicular to the strike joints and was also dipping nearly vertically. Spacing of these joint sets typically varied between five and ten feet.

In excess of 35,000 cubic yards of rock had to be excavated by the project contractor within the three-month summer recess to minimize the impacts on students and faculty. The project specification required the blasting subcontractor to:

- Forward for review details of the subcontractor's proposed blast rounds.
- Conduct pre-blast condition surveys on the adjacent structures.
- Maintain blast vibration within a set of limits at the nearest structures (see Table 2).
- Maintain air blast overpressure at less than 0.014 psi at the nearest structures.
- Conduct blast vibration monitoring for every blast round.

In addition, provisions were included in the project specification to minimize the potential for rock block movement, including:

- A maximum ten-foot rock cut depth when blasting within 50 feet of adjacent structures.
- A requirement to muck out after every round, or pull blasted rock away from the face, in order to maintain a free face.
- Requirements for grouted dowels at the excavation limits in the area of adjacent buildings (see Figure 14) that would be

FIGURE 12. Looking southwest towards Stimpson Hall (to the left) and Olin Library (to the right).

FIGURE 13. Looking southeast, Stimpson Hall is to the right, with underpinning and needle beam at the proposed stairwell shaft. The soil nailing earth support system is visible beyond Stimpson Hall (to the left).

TABLE 2
Maximum PPV
for Underground Library Addition

Distance (ft)	PPV (in/sec)
> 20	2
10 to 20	3
< 10	4

installed prior to blasting in the area.

- Requirements for prestressed rock anchors installed into the rock face at the excavation perimeter as the excavation was carried down at the perimeter (see Figure 14).
- A requirement for careful perimeter control blasting.

The blasting subcontractor started blasting at the interior of the site where the rock cuts were shallowest and where the distance to the nearest structure was the largest. The subcontractor utilized a four- by five-foot drill pattern with three-inch diameter holes for much of the excavation. A non-electric delay detonation system was used, with two- by 16-inch cartridges of a high-energy dynamite formulated for trenching and submarine applications. Figure 15 illustrates a typical hole loading at the interior of the site. Generally, four decks (explosive charges separated from other explosive charges in a blasthole by stemming) were used in each hole, with each deck on a separate delay in order to allow full depth rock excavation while minimizing charge weight per delay.

As the blasting subcontractor approached Stimpson Hall and Olin Library, a face parallel to each building was developed. Once within 20 feet, the subcontractor changed to a three- by three-foot pattern with up to five decks per hole, each on a separate delay. Figures 12 and 13 offer views of the excavation face approximately 20 feet from Stimpson Hall, and approx-

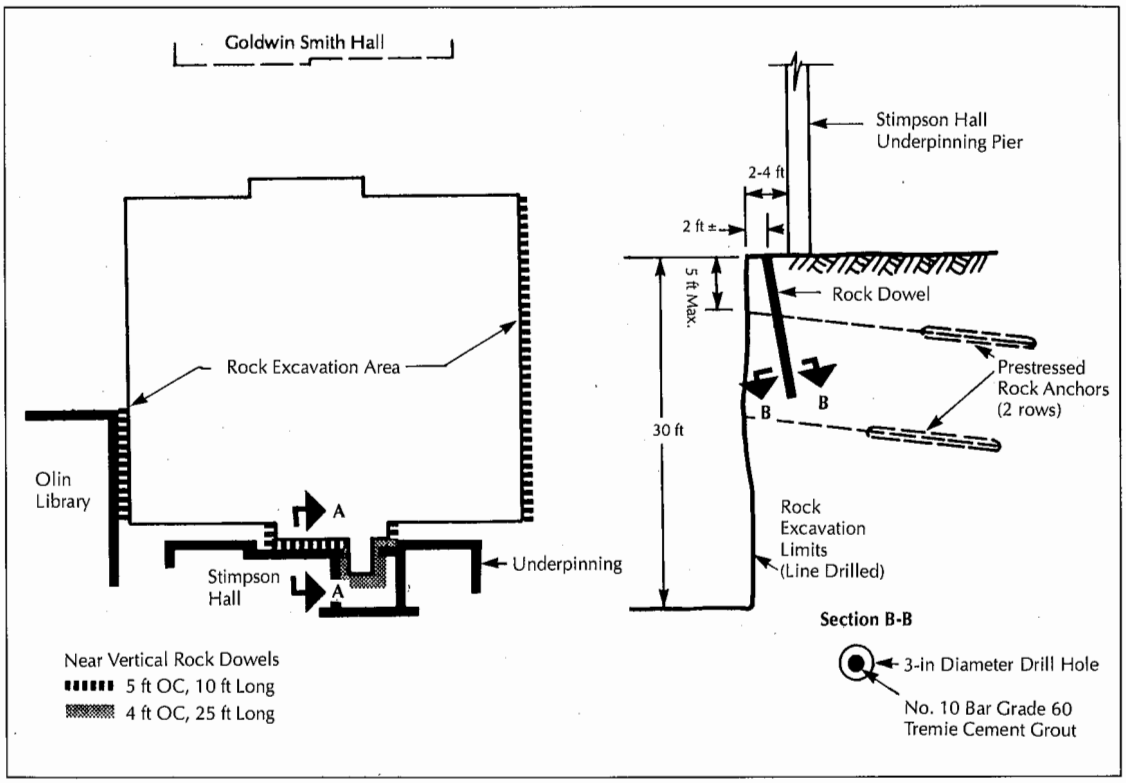

FIGURE 14. Grouted dowel and anchor locations are shown to the left; a typical section of the rock dowel and rock anchor is shown to the right.

imately 25 feet from Olin Library. The non-electric delays are laid out. Figure 13 also shows that the muck pile was pulled away from the face in order to ensure a free face for the next round.

At the perimeter of the excavation near Stimpson Hall, a modified line drilling method was used in the area of the needle beam, where the rock bench was only about 0.5 to four feet wide. Three-inch diameter holes were drilled in these areas at six to eight inches on center, and every third hole was loaded with four to five decks of 1.5-inch diameter dynamite explosive, with a loading density of about 0.10 to 0.15 lb/ft. These perimeter holes were detonated after production blasting had been completed in order to remove a final three-foot wide bench. The results of this combination line drill/cushion blasting method were generally good, as indicated on the left side in Figure 16.

At the right side of Figure 16, and at other areas where bench widths were four to eight feet or more, pre-splitting or cushion blasting was allowed at the perimeter. Cushion blasting was generally used, with perimeter holes drilled at 18 to 24 inches on center. Every other hole was loaded with 1.5-inch dynamite explosive, with an average loading of about 0.20 to 0.40 lb/ft. Cushion blast holes were generally decked with four separate delays per hole in order to minimize resulting vibrations at the adjacent structure, and were fired either after production blasting in the area or along with the final row of production holes. There was some overbreak and damage to the remaining rock with the cushion blasting, especially at outside corners. Although there was some localized block movement near the top of the rock in some areas, the combination of rock dowels and adequate bench width prevented rock block movement below the building foundations or earth support system in these cushion blast areas.

One area where rock block movement was of particular concern was at the outside corners of the 35-foot deep rock cut inside Stimpson Hall for the proposed entrance stairway (see Figure 17). There was concern that there may be the possibility of a dip joint intersecting one of the corners, which would result in overbreak or block movement under the underpinned foundations. For this reason, additional near

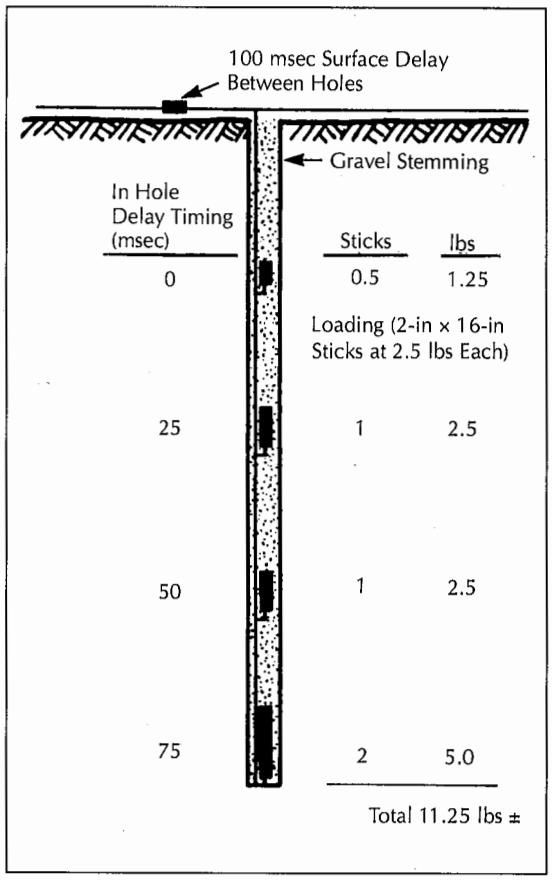

FIGURE 15. Typical loading for the 30-foot deep production holes (interior of site).

vertical dowels 25-feet long were installed in this area prior to blasting. Prestressed rock anchors were also installed as the rock cut limits were exposed (see Figure 14). Unfortunately, at the right (west) side corner, explosive gasses from a production round inside the building entered and opened a near vertical "dip" joint extending under the corner of the underpinning, resulting in small outward movement of a large slab of rock. Figure 18 shows a close-up view of this area. A small rock block was removed at the top of the rock (see Figure 17). Additional block movement and/or overbreak was prevented by the pre-installed rock dowels and one rock anchor that had been installed. Two additional rock anchors were installed to stabilize this rock block prior to proceeding with further work.

Additional block movement/overbreak resulted from explosive gasses entering "dip"

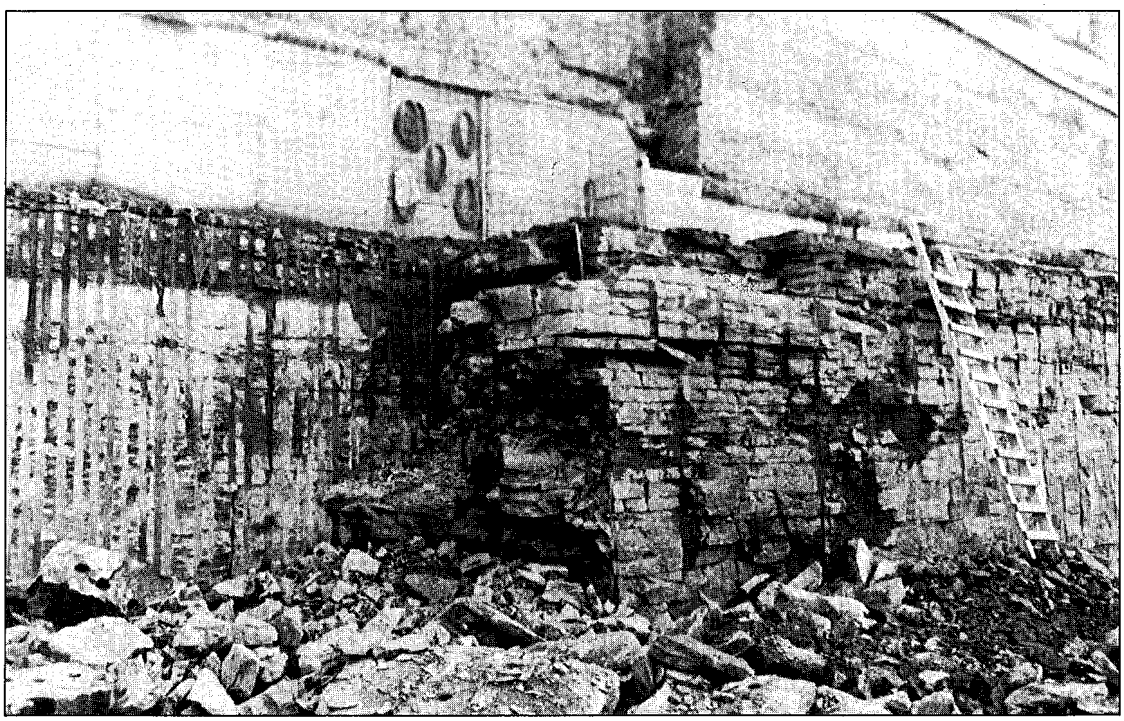

FIGURE 16. A view of the excavation limits at Stimpson Hall, showing line drilling results at left, pre-splitting at right. Note the damage to the outside corner.

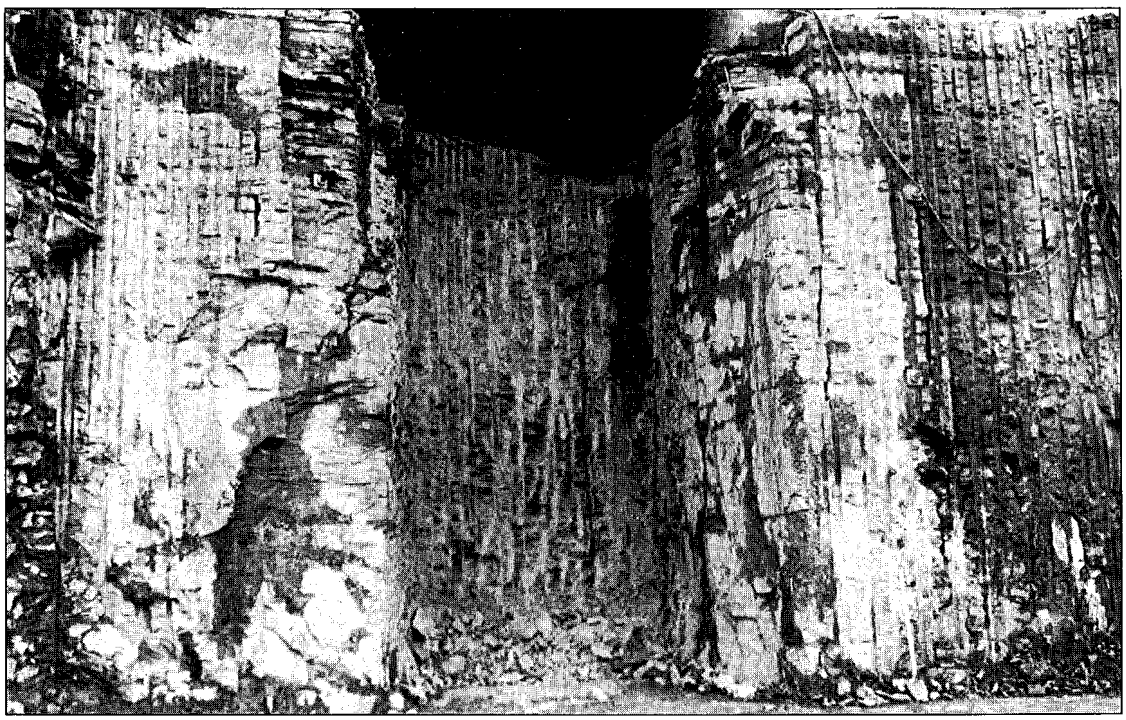

FIGURE 17. A view of the excavation limits at the stairway area that was blasted inside Stimpson Hall, showing the opened "dip" joint at the right side corner. A rock block at the top was removed from this area.

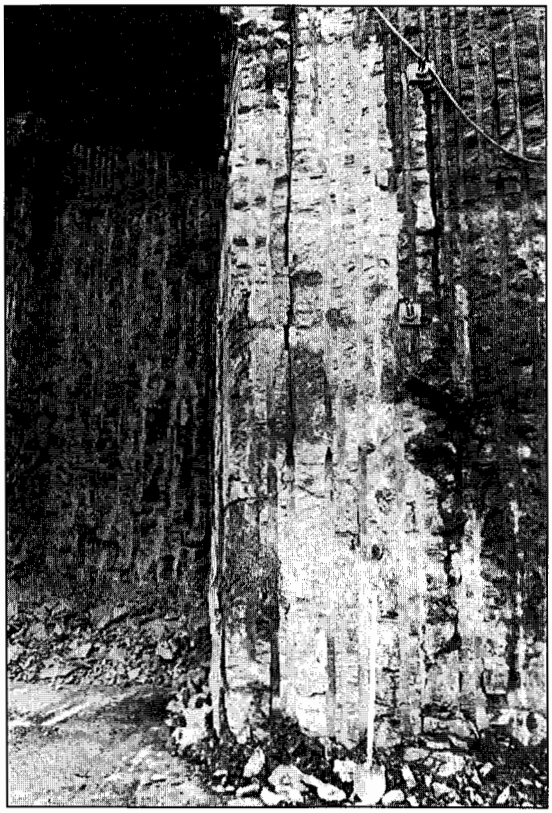

FIGURE 18. A photo of the opened "dip" joint that caused the block movement at the corner of the stairway area inside Stimpson Hall.

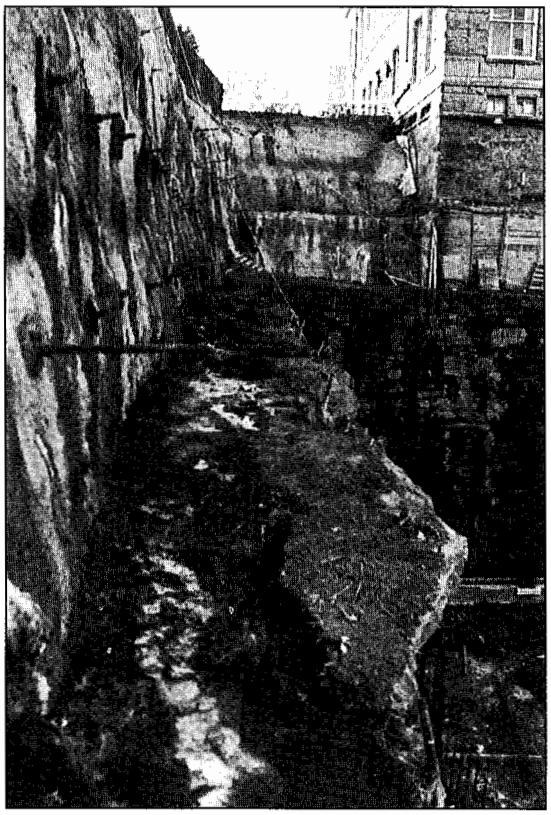

FIGURE 19. Rock block movement along the "dip" joint at the east side of the excavation. The soil nailing earth support is above the rock bench.

joints subparallel to the east side of the excavation, below the soil-nailed earth support wall (see Figure 19). Again, a combination of the grouted pre-installed dowels and an adequate bench width avoided problems from rock block movements that might otherwise have extended beneath and undermined the soil slope.

Vibrations at Stimpson Hall were monitored for each round with a conventional digital seismograph, and recorded vibrations were generally below the specified limits. When blasting within ten feet of the structure, the maximum recorded PPVs generally ranged from about two to three in/sec at recorded frequencies in excess of 100 Hz. No damage was found within the adjacent buildings.

Additional vibration measurements were made during the close-in blasting by Cornell University, using digital equipment operating at a sampling rate many times greater than the conventional digital seismograph. The recorded PPVs were generally higher than those recorded on the conventional seismographs, which sample at a rate of 1,000 to 2,000 samples per second. Fast Fourier Transform analyses performed on the waveforms obtained by the Cornell University equipment indicated that the predominant frequency of blasting vibrations from close-in blasting (within 40 to 50 feet) was at about 330 Hz, and with another predominant frequency at about 140 Hz (see Figure 20). Thus, it was apparent that conventional seismographs with frequency ranges up to only 100 to 200 Hz were not measuring some of the high frequency energy from the close-in blasting, thus resulting in recording PPVs lower than they would actually be.

At first glance, this situation would appear to be a troublesome finding, since it can be assumed that at many similar close-in blasting

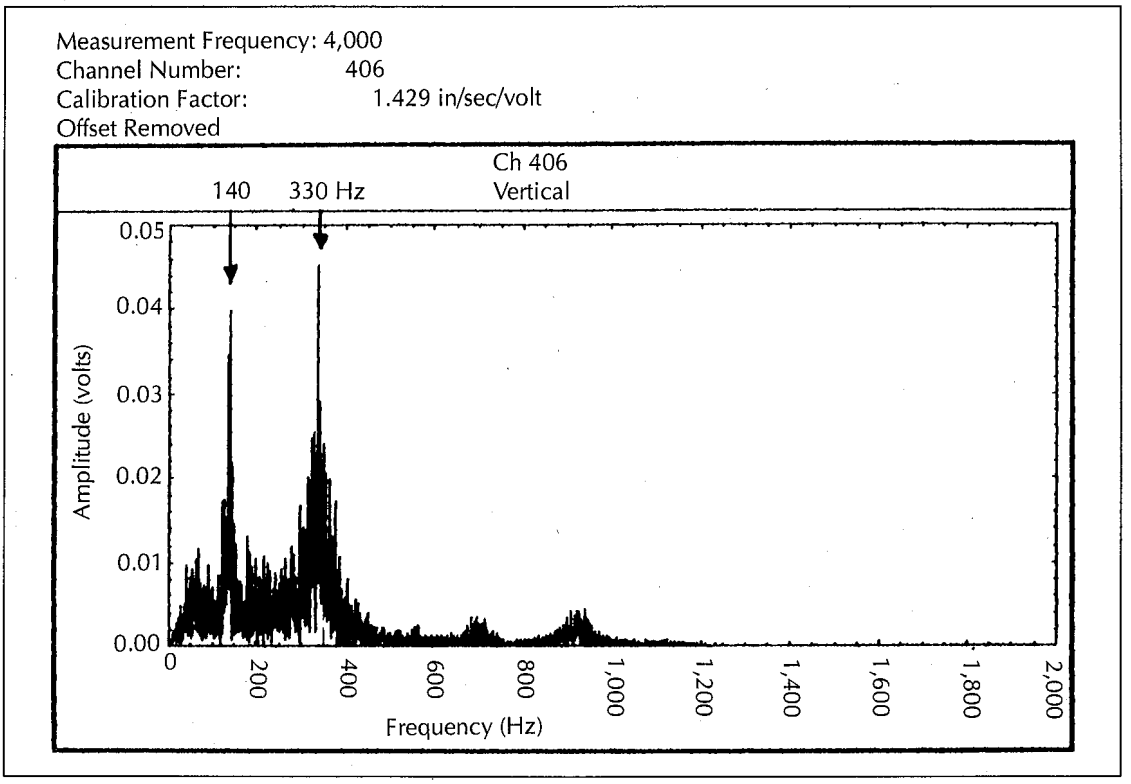

FIGURE 20. Fast Fourier Transform analysis of a blast round 25 feet from Stimpson Hall, showing the predominant frequencies at 330 and 140 Hz.

projects using conventional seismographs, the recorded PPVs may be less than the actual due to the loss of high frequency energy. However, if the displacements associated with these high frequency vibrations are viewed, it can be seen why there was no vibration-related damage for this and most similar close-in blasting.

As an example, a blast was selected for analysis that was detonated approximately 25 feet from Stimpson Hall. The maximum PPV at the Stimpson Hall foundation recorded by the conventional seismograph was approximately 1.6 in/sec at a frequency of about 130 Hz. Results with the Cornell University equipment yielded a PPV of 2.2 in/sec, an increase of over 35 percent, at a predominant frequency of 330 Hz.

Both these results are plotted on the Bureau of Mines alternate "safe limit" criteria plot (see Figure 21). Although the 2.2 in/sec PPV exceeds the so-called "safe limit" for residential structures of 2.0 in/sec (above 40 Hz), the calculated displacement is well below the Bureau's displacement criteria of 0.008 inches (for 10 to 40 Hz). In fact, the calculated displacement for 2.2 in/sec at 330 Hz is only 0.0011 inch, which is even less than that calculated for 1.6 in/sec at 130 Hz (0.0016 inches). Although high frequency vibrations may result in some energy being missed by conventional seismographs, the high frequency vibrations (even with high PPVs) result in very small, elastic displacements that are not generally damaging to structures.

Of far more concern when blasting close-in to structures is the potential rock block movement that was illustrated at the Cornell University project that resulted from explosive gasses entering joints in seams in the rock mass. The resulting permanent ground deformations can be several orders of magnitude larger than displacements from the elastic ground vibrations, and can result in loss of support for, or damage to, adjacent structures.

Summary & Conclusions

The case histories demonstrate that, with some

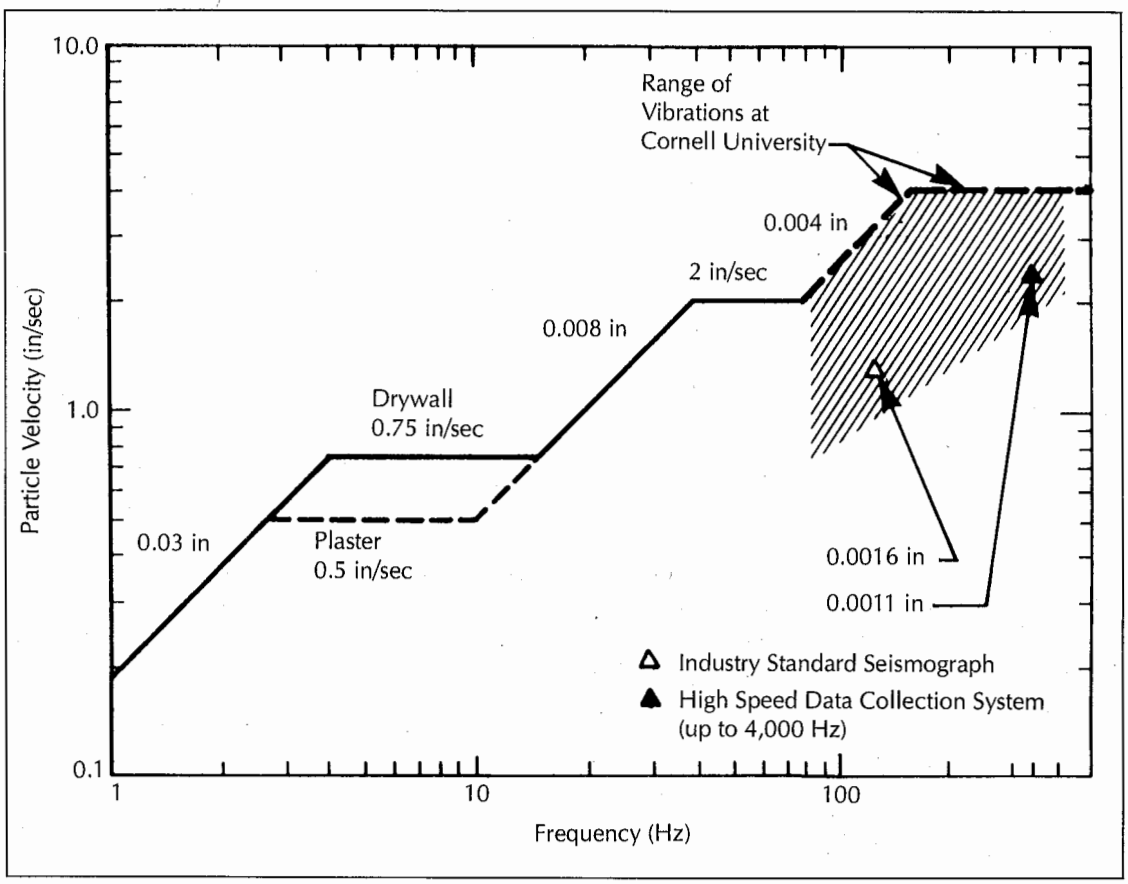

FIGURE 21. Bureau of Mines "safe limit" criteria with the Cornell University blast vibration data, showing low displacements at high frequencies.

appropriate precautions, blasting can safely be performed in close proximity to adjacent above ground structures and below ground utilities. In order to design the appropriate specification criteria and safety precautions, it is important to:

- Look at each situation and make judgments as to which mechanism of blast-induced ground motion poses the greatest threat to the structure or pipeline.
- Determine the type, age and condition of the structure or pipeline.

While blast vibration is often the only mechanism considered, permanent ground deformations caused by cratering or rock block movement can often pose a greater threat to a structure. By using simple techniques such as grouted dowels installed prior to blasting, and allowing adequate relief for each round, these potential hazards can be overcome.

When blasting close-in to structures bearing on rock, ground vibration frequencies are likely to be over 100 Hz and often much higher. As a result of these high frequencies:

- Amplification of vibrations within structures is generally not a problem, and displacements are low. At the Cornell University project, PPVs over four in/sec were recorded at the foundation with frequencies in excess of 200 Hz, and there was no apparent damage. The estimated ground displacements were less than 0.004 inches, which was less than the Bureau of Mines displacement limit of 0.008 inches between 10 and 40 Hz.
- A significant amount of high frequency energy (above 150 to 200 Hz) may be missed by conventional seismographs

due to their relatively low frequency response and sampling rates. However, these high frequency vibrations (even with high PPVs) result in very small, elastic displacements that are not damaging to structures in general.

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ANDREW F. MCKOWN *is a Senior Specialist with Haley & Aldrich, Inc., in Cambridge, Massachusetts. He received a B.S. in civil engineering from Tufts University in 1969 and an M.S. from the Massachusetts Institute of Technology in 1978. His experience includes a wide variety of geotechnical projects, with particular emphasis on controlled blasting, construction vibrations and rock mechanics. He is a former member of the Explosives Advisory Committee to the Massachusetts Board of Fire Prevention, a member of the Program Committee of the Society of Explosives Engineers, the ASCE Rock Mechanics Committee, the International Society for Rock Mechanics and the Underground Technology Research Council. He is a registered professional engineer in Massachusetts.*

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Tunnel Boring Machine Excavation of the Beverly Sewer Tunnel

A refitted tunnel boring machine can operate effectively in hard, high-strength igneous rock, offering cost and time savings and reduced potential for vibration damage and complaints.

GEORGE W. HARTNELL, III, &
ANDREW F. MCKOWN

While tunnel boring machines (TBMs) have been used world-wide for decades, the first successful use of a tunnel boring machine in hard rock in Massachusetts occurred in 1990 at the Beverly Sewer Tunnel in Beverly. Many factors were involved in the selection of mechanical excavation over conventional drill-and-blast tunneling. The performance of a reconditioned TBM that completed the tunnel had a beneficial impact on the cost, scheduling, ground support requirements and environmental issues that were involved with the Beverly Sewer Tunnel project.

Project Description

The Danvers-Beverly Relief Interceptor Project. The South Essex Sewerage District (SESD), servicing the communities of Danvers, Beverly and Salem, began conceptual planning of the relief interceptor in the mid-1970s in order to create a regional wastewater system that combined functional existing interceptors, some of which are a century old, with new relief interceptors. The additional sewer capacity would thereby provide for future development and population increases. Increased capacity would also eliminate overflows that degrade the Beverly Harbor and Danvers River Estuary and jeopardize the local commercial fishing industry.

As shown in Figure 1, the Danvers-Beverly Relief Interceptor connects the Danvers Pump Station to the proposed Beverly Pump Station, which in turn transports the sewage to the SESD Treatment Plant. The project involved a number of phases:

- Cut-and-cover construction from the Danvers Pump Station to the Bass River;
- Extending the interceptor beneath the river (Bass River Siphon) to a junction chamber at the Bass River Shaft;
- Tunneling beneath the Goat Hill area of Beverly (Beverly Sewer Tunnel) to the Water Street Shaft;
- Cut-and-cover construction from the Water Street Shaft to the Beverly Pump Station; and,

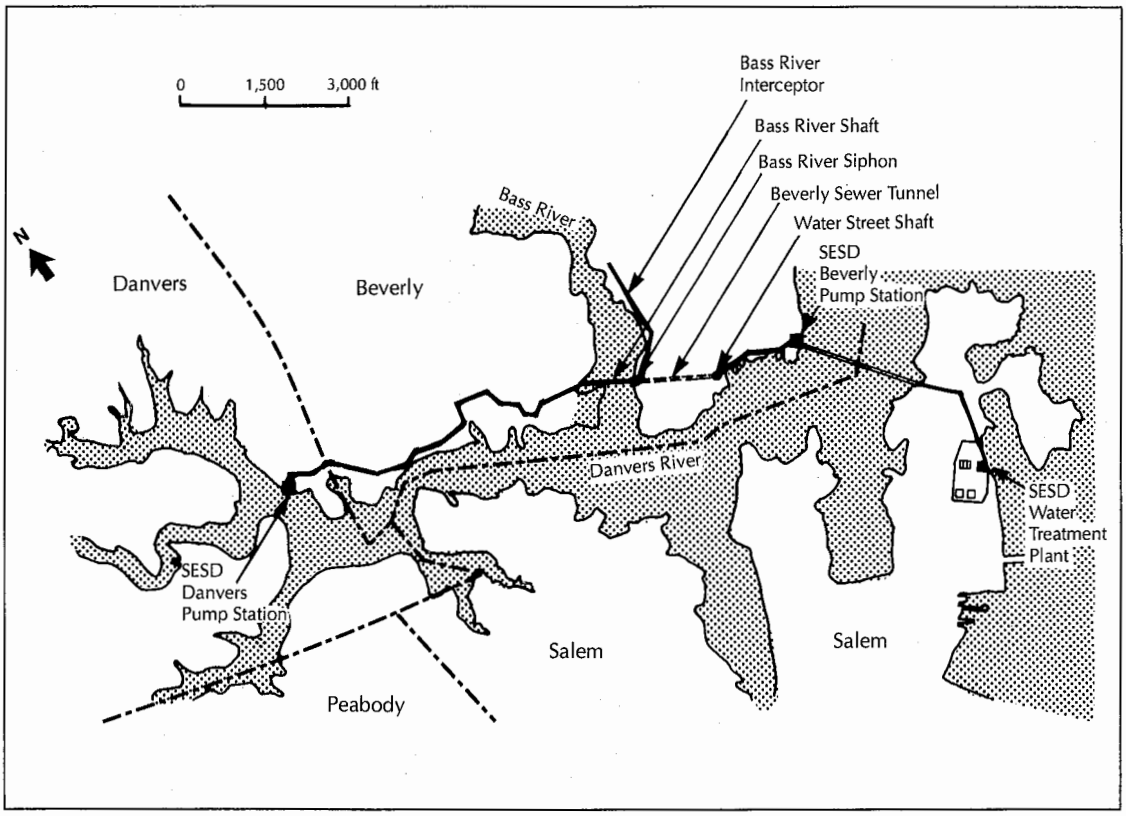

FIGURE 1. A schematic plan of the Danvers-Beverly Relief Interceptor location.

- Extending the interceptor beneath Beverly Harbor to the SESD Treatment Plant in Salem.

The tunneled portion of the project in Beverly, from the Bass River Shaft under Goat Hill to the Water Street Shaft, called the Beverly Sewer Tunnel, is under discussion here.

The Beverly Sewer Tunnel. The Beverly Sewer Tunnel was designed to combine incoming flows from the Bass River Siphon and the Bass River Interceptor and convey them to the Beverly Pump Station (see Figure 1).

Construction of the Beverly Sewer Tunnel required the excavation and support of two shafts through soil overburden and rock to a depth of approximately 60 feet, and approximately 1,570 feet of 8.5-foot diameter hard rock tunnel. The location of the shafts and the tunnel alignment are shown in Figure 2.

From a hydraulics standpoint, the tunnel and shafts were designed as an "inverted siphon." A junction chamber at the top of the Bass River Shaft combines the inflows, which then pass through a weir system and into one or more of three ductile iron sewer pipes with outside diameters of 25.8 inches, 32 inches and 44.5 inches. The pipes, flowing full, carry the flow down the shaft, through the tunnel, and up the Water Street Shaft. An outlet chamber at the Water Street Shaft diverts the flow into a 54-inch diameter sewer pipe that leads to the pump station. The tunnel and shafts were backfilled with lean concrete in order to avoid having to install permanent rock support and to provide some additional corrosion protection for the ductile iron pipes.

Site and Subsurface Conditions. Located approximately 25 miles northeast of Boston, the city of Beverly is in the Seaboard Lowland Section of the New England Physiographic Province. Rock in the region is typically classified as igneous or metamorphic, from the Paleozoic Era. It has been subjected to several episodes of tectonic activity during which large-scale regional folding and faulting have developed ac-

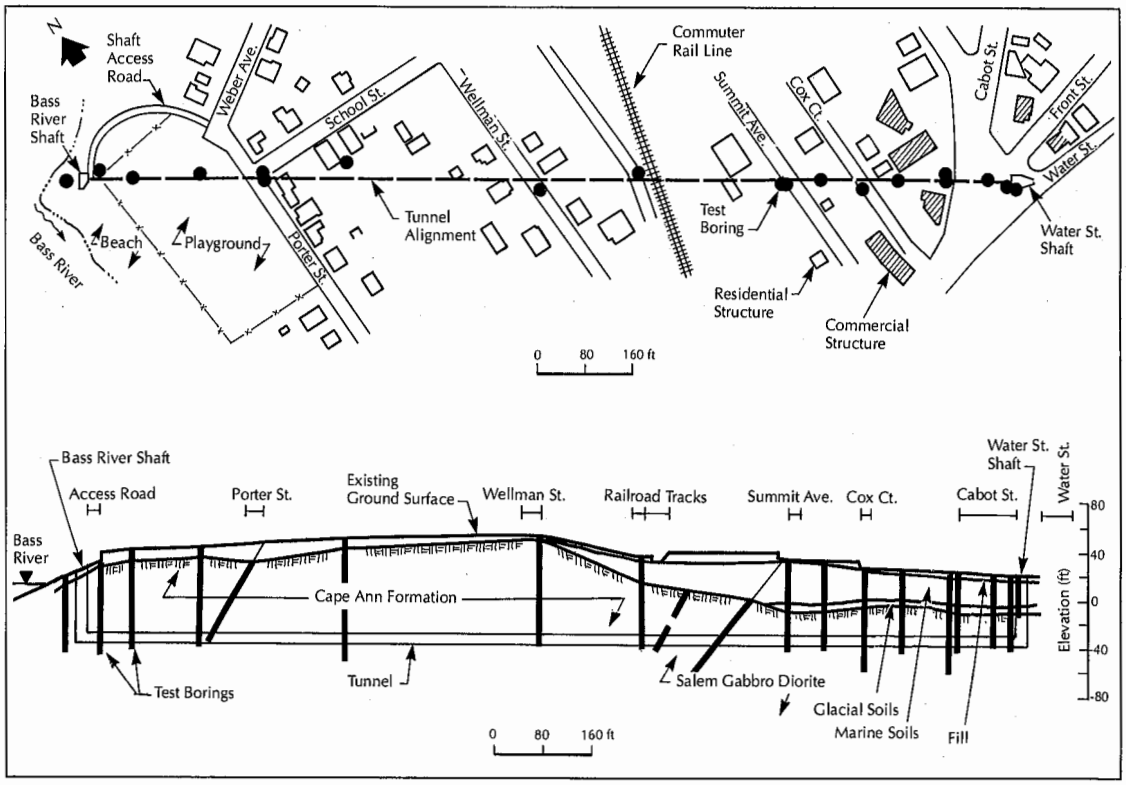

FIGURE 2. Plan and general geologic section of the Beverly Sewer Tunnel location.

companied by granitic intrusions.[1]

A detailed subsurface exploration program consisting of test borings, field permeability testing, seismic refraction surveys and geologic field mapping was conducted in 1983 to determine stratigraphy, lithology and rock mass properties along the tunnel alignment.[2,3] Laboratory tests and analyses were performed on soil and rock samples to aid in classification and to define significant engineering properties.

Figure 2 shows a simplified geologic profile along the tunnel alignment. Overburden soils ranged in thickness from about two to 42 feet. The overburden typically consisted of marine deposits of fine sands and clayey silts overlying glaciofluvial and glacial till deposits.

Two major bedrock units were encountered along the tunnel alignment: Cape Ann Granite and Salem Gabbro-Diorite. The granite is described as hard to very hard, very slightly weathered, coarse to medium grained. The gabbro-diorite is described as hard to very hard, slightly weathered, medium to fine grained. Table 1 illustrates the average mineralogical composition of the two rock types. Based on the exploration program, it was anticipated that about 60 to 65 percent of the tunnel excavation would be in the Cape Ann Granite formation, about 30 to 35 percent in the Salem Gabbro-Diorite, and the remaining in mafic igneous dike rock.

Table 2 lists some average engineering properties of the intact rock derived from laboratory testing of core samples. Based on Deere and Miller, the intact Cape Ann Granite would be classified as high-strength, average modulus rock; while the Salem Gabbro-Diorite would be classified as high-strength, high modulus rock.[4]

During geologic field mapping four major joint sets were identified in the granite, as summarized in Table 3. No outcrops of gabbro-dioritic rock were exposed to allow mapping. However, based on analyses of rock core from test borings, including an angled boring in the gabbro-diorite, it was assumed that the primary joint sets were similar throughout the alignment.

TABLE 1
Mineralogical Composition From Petrographic Analyses

Cape Ann Granite

Mineral	Estimated Percentage
Microperthite	60-70
Quartz	20-30
Hornblende	0-5
Pyroxene	0-5
Opaques & accessories	0-5

Salem Gabbro-Diorite

Mineral	Estimated Percentage
Plagioclase	50
Quartz	15-20
Hornblende	15-20
Sericite	10
Biotite	5
Augite	5
Opaques & accessories	0-5

From Refs. 2 & 3

Several shears (naturally occurring rock discontinuities along which displacement of limited extent has occurred) were observed during the geologic mapping program. These shear features were generally continuous, but were not considered to be shear zones (*i.e.*, the shearing was confined to a plane and did not significantly affect the rock on either side of the plane). The exploration program did not identify any discontinuities that could be classified as faults.

Rock mass quality along the proposed tunnel alignment was determined using the Rock Quality Designation (RQD) technique.[5] RQD is defined as the summation of intact, unweathered pieces of rock four inches in length or greater within a core run, divided by the total length of the core run, expressed as a percentage. Within the tunnel's zone of influence (invert to one tunnel diameter above crown), approximately 93 percent of rock cores obtained during the subsurface exploration program had RQD values exceeding 50 percent, with a mean of about 73 percent. This rock would be classified as fair to good quality according to Deere *et al.*[5] RQD in the gabbro-diorite was typically slightly higher than in the granite.

Groundwater levels were typically about 47 to 75 feet above the tunnel invert and were consistent with variations in topography and ground surface. Based on water pressure tests in borings along the tunnel alignment, the rock mass generally exhibited low equivalent permeability (less than 10^{-5} cm/sec) with some isolated areas of potentially higher water inflows. Total inflow within the tunnel was estimated to be less than 300 gallons per minute (gpm), with each shaft adding an estimated additional 20 gpm.

Design Requirements

The vertical alignment of the Beverly Sewer Tunnel was established to provide a minimum of one tunnel diameter of rock cover above the crown of the tunnel. The minimum cover occurred near the Water Street Shaft (east) end of the tunnel (see the profile in Figure 2). The tunnel sloped up at a slope of about one foot in 100 feet from the designated Bass River work

TABLE 2
Summary of Engineering Properties of Intact Rock

Property/Parameter	Average Value: Cape Ann Granite	Average Value: Salem Gabbro-Diorite
Unit Weight (pcf)	164	179
Total Hardness	183	107
Compressive Strength (psi)	20,200	18,200
Tangent Modulus (psi)	9.8×10^6	11.3×10^6

From Refs. 2 & 3

TABLE 3
Summary of Cape Ann Granite Joint Sets

Joint Set No.	Strike Range	Dip Range	Typical Spacing (ft)
1	N38°E - N90°E	46° - 90° SE	1-3
2	N40°W - N10°E	49° - 90° NE 77° - 90° SW	1-3
3	N40°W - N90°E	71° - 90° NE 71° - 90° SW	> 10
4	N73°E - N20°W	17° - 34° NE	> 10

From Ref. 3

shaft towards the Water Street Shaft in order to allow drainage away from the heading as tunneling advanced. Tunneling was only allowed from the Bass River Shaft, which was separated from the residential community by a playground (see Figure 2), in order to reduce construction impacts on the community and to minimize traffic disruption in the congested streets around the Water Street Shaft.

It was assumed during the design phase work, which was completed in 1983, that the tunnel would be driven using drill-and-blast techniques due to economic considerations, although provisions were included in the contract documents to allow TBM excavation as an alternative. The contractor was given flexibility to choose the tunnel cross section geometry within the minimum and maximum limits indicated in Figure 3.

Tunnel Support. Fully grouted rock dowels were required as minimum initial support at the tunnel portals at the Bass River and Water Street shafts. These dowels were approximately 15 feet long and were installed as pre-reinforcement — *i.e.,* they were installed prior to commencing tunnel excavation from the Bass River Shaft and prior to holing through at the Water Street Shaft. Portal pre-reinforcement consisted of a single row of four or five dowels (No. 8, upset threaded bar, grade 60 steel, about 15 feet long) located two feet above the crown of the tunnel, angled upward at 30 degrees from horizontal and parallel to the tunnel centerline.

Based on the relatively good quality of the rock mass and the favorable orientation of the primary joint set (striking nearly perpendicular to the tunnel alignment), there was no minimum initial rock support specified for the tunnel. Instead, the contract documents required

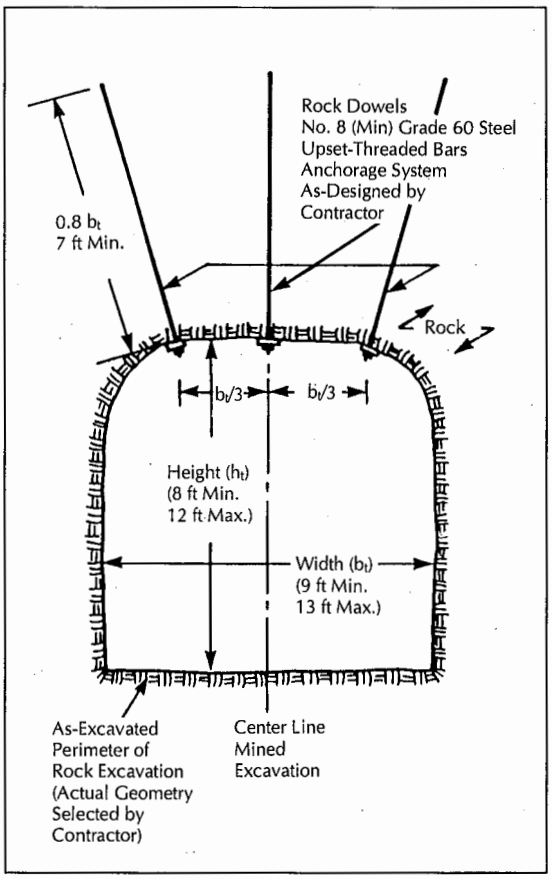

FIGURE 3. Schematic section showing minimum/maximum specified tunnel size and suggested initial support.

FIGURE 4. Looking southeast along the tunnel alignment from the playground at the Bass River Shaft.

the installation of rock support on the basis of observed rock conditions in the tunnel. Figure 3 shows suggested initial rock support where required, consisting of untensioned, fully grouted rock dowels, three per row, spaced at five-foot intervals along the tunnel axis. Based on the available geotechnical information for the site, it was estimated that up to 20 percent of the tunnel drive would require pattern rock dowels for initial rock support, assuming smooth blasting techniques were used at the tunnel crown.[3] It was anticipated that spot dowels would be required to support isolated unstable rock blocks at random locations throughout the tunnel.

Schedule. It was estimated that tunnel excavation, assuming drill-and-blast methods, would advance at the rate of about six feet (or one tunnel round) per day, for a total duration of about 262 work days, or about 45 weeks (at six days per week, 12 hours per day). The contractor was to complete all work within 540 calendar days (77 weeks) after formal execution of the contract agreement.

Construction Impact Mitigation. The distance from the crown of the tunnel to ground surface ranged between 45 and 79 feet. Above the tunnel was a residential and light commercial neighborhood that contained a number of historical buildings, some dating back to the seventeenth century. Figure 4 shows some of the houses over the tunnel alignment. It was taken from the playground adjacent to the Bass River Shaft, looking southeast towards a drill rig installing geotechnical instrumentation over the crown of the tunnel. Figure 5 presents a view looking northeast at the Water Street Shaft, and depicts some of the commercial and residential structures near the shaft.

As noted previously, it was assumed during

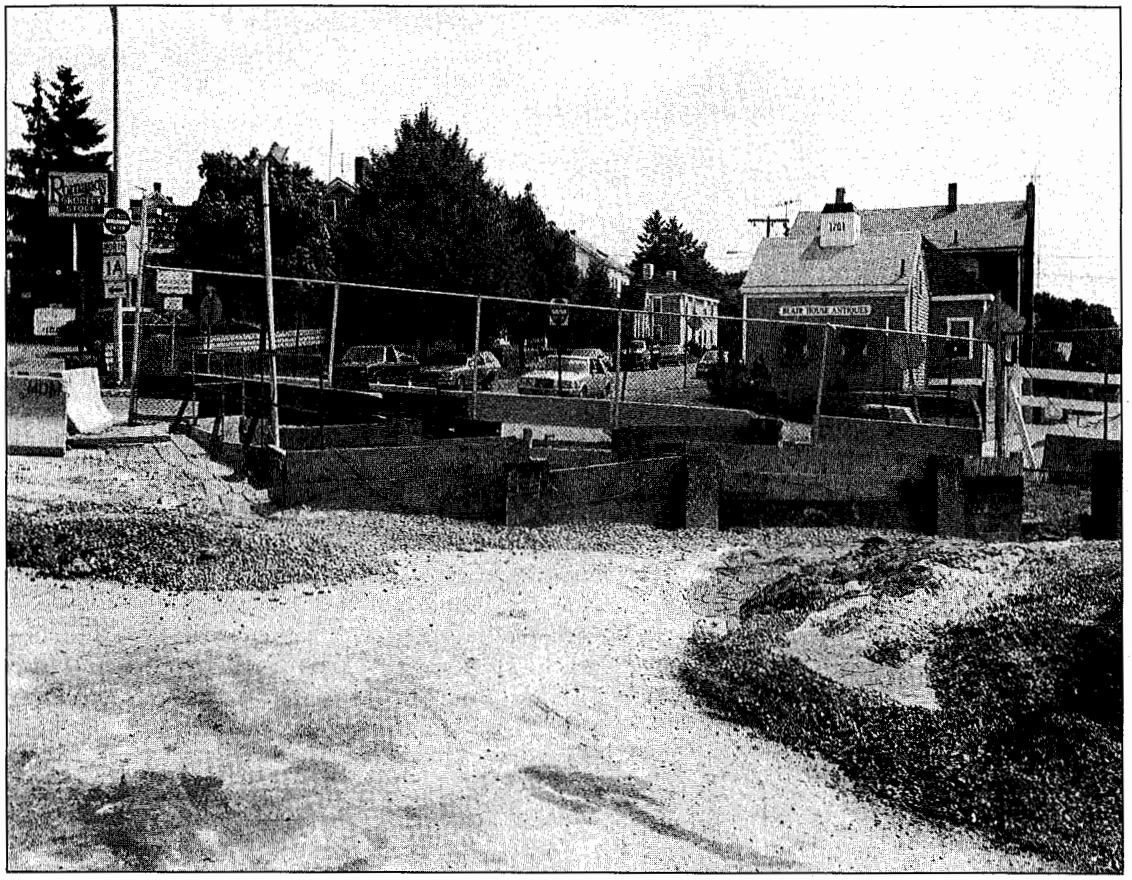

FIGURE 5. Looking northeast at the Water Street Shaft and nearby historic structures.

the design phase work that the tunnel would be driven using drill-and-blast techniques. Because of the proximity of the residences to the shaft and tunnel construction, in particular to blasting operations, the impacts of ground vibrations and air blast were sensitive issues within the community. A number of provisions were incorporated into the contract documents to minimize these and other construction related impacts on people living and working in the area.

Blasting Impact Mitigation. Some of the blasting related mitigation provisions included:

- Restrictions on blasting hours to the period between 8:00 a.m. and 7:00 p.m., Monday through Saturday.
- Limits on the maximum peak particle velocity of ground vibration and maximum peak air blast overpressure at the nearest residential structure, based on United States Bureau of Mines research on blasting near residential structures (see "Close-In Construction Blasting: Impacts & Mitigation Measures," on pages 73–92 for a more complete discussion of these limits).[6,7]
- Vibration monitoring and reporting for each blast by an independent professional engineer or seismologist under contract to the contractor.
- Pre-Blast Condition Surveys of all above ground structures in the area of blasting, by an independent professional engineer or seismologist under contract to the owner.
- Pre-Blast Radon Surveys of all residential structures in the area of blasting, by an independent engineering consultant specializing in such work, under contract to the owner. This provision was added as a result of citizen concerns about tunnel

TABLE 4
Comparison of TBM *vs.* Drill-&-Blast Tunneling

TBM Tunneling	Drill-&-Blast Tunneling
Continuous Operation	Cyclic Operation
High Initial Capital Cost	Relatively Low Initial Capital Cost
Potentially Long Delivery Time	Relatively Short Delivery Time
Circular Profile Only	Variable Shape & Size Openings
Excavation Rate: Relatively High in Favorable Geology Poor in Unfavorable Geology	Excavation Rate: Relatively Low Adaptable to Adverse Geology
Relatively Low Induced Ground Vibrations	Ground Vibrations & Overpressures
Little Rock Disturbance; Minimal Overbreak; Rock Support Requirements Potentially Reduced	Rock Disturbance; Greater Rock Support Requirements
Uniform, Small Muck Chips	Bulky Blast Rock

From Ref. 8

excavation increasing the migration of radon into basements of houses above the alignment.

- Submittal of detailed blasting plan by the contractor, for review by the design engineer, prior to blasting at either shaft.

Citizen Mandated Mitigative Measures. In addition to the blasting related controls, there were also provisions to minimize noise impacts and disruption and danger from truck traffic through the residential neighborhood. As a result of meetings between the owner, engineer, city officials and a citizens' group, the following provisions were added to the contract documents:

- Overall operations only permitted between 7:00 a.m. and 8:00 p.m., Monday through Saturday. In addition, the contractor was *required* to work six days a week (Monday through Saturday) in order to reduce the total duration of construction and so reduce the total duration of related impacts on residents.
- Prohibition of hauling of muck from the Bass River work shaft through adjacent residential streets. All muck was loaded onto a barge in the adjacent Bass River using a conveyor system and barged to an offloading site away from the residential neighborhood.
- Prohibition of any truck traffic through residential streets on week days during the school year between the hours of 7:30 a.m. to 8:15 a.m., and between 2:15 p.m. to 3:00 p.m.
- Prohibition of contractor employee parking on streets near the Bass River work shaft. Only a limited staging area was provided at the Bass River work shaft, so the contractor was required to provide off-site office facilities and staging area for workers and materials.

In addition to the above provisions, presentations were made to residents prior to construction to address their concerns and to inform them of the work to be undertaken and the controls in place to minimize impacts on them. During tunnel construction, informational meetings were held with residents on a monthly basis to keep them informed of progress and to answer questions or respond to complaints.

Geotechnical Instrumentation & Monitoring

A program of geotechnical instrumentation and monitoring was undertaken by the owner's geotechnical consultant during construction and included:

- Observation wells and piezometers to monitor and document the effect of construction on the groundwater table, as related to water well yields and long-term effects of groundwater drawdown on adjacent structures.
- Settlement pins on structures to monitor and document the effects of construction

on adjacent structures.

- Borros settlement points to monitor surface settlements that could have resulted from drawdown within marine deposits.
- Multiple Position Borehole Extensometers (MPBXs) installed from ground surface to monitor rock movements above the crown of the tunnel, in order to evaluate the performance of the initial rock support system, especially in areas of minimal rock cover.
- Blast monitoring to observe compliance with submitted procedures and supplement and confirm the contractor's vibration measurements.

Shaft & Tunnel Construction

Due to a lack of funding, the project, which was designed in 1983, did not go to bid until 1989. The construction contract was awarded to the low bidder in the fall of 1989. The contractor proposed using a reconditioned TBM to excavate the tunnel instead of employing drilling and blasting. However, drill-and-blast excavation was still required for the excavation of the shafts and a starter tunnel.

TBM vs. *Drill-and-Blast Tunneling*. The advantages and disadvantages of TBM *vs.* drill-and-blast tunneling are compared and summarized in Table 4. When construction of the Beverly Sewer Tunnel was released for bid, it was anticipated that because of the relatively short length of the tunnel (slightly less than 1,600 feet), it would most economically be excavated by conventional drill-and-blast tunneling. It was felt that the cost of mobilizing a TBM would be difficult to amortize on such a small project. In addition, it was felt that the relatively high compressive strength, high hardness and high quartz content in the granitic and gabbro-dioritic rock would result in relatively low penetration rates and require the frequent maintenance and replacement of cutters.

The contractor judged that a properly configured TBM could advance at significantly higher rates than with drill-and-blast tunneling. The contractor estimated that even with the time to mobilize, assemble, demobilize and disassemble the machine, there would be savings in construction time and, therefore, in construction costs over drill and blast. The contractor also expected to see reduced costs (over drill and blast) from:

- Reducing rock support requirements as a result of the minimal rock disturbance by the TBM.
- Reducing the amount of muck to remove and concrete backfill to place as a result of a reduction in overbreak.
- Reducing the time and fees for the blast monitoring consultant, whose presence would only be required during the shaft and starter tunnel blasting.
- Reducing the potential for damage claims as a result of blasting vibrations.
- Reducing potential work stoppages or restrictions as a result of community complaints or actions.

The contractor planned to lease a used TBM that would be available after a short reconditioning period, so two of the biggest disadvantages of TBM tunneling were immediately offset (*i.e.*, high initial capital cost and long delivery time). Thus, the combination of TBM leasing, along with the potential cost savings noted above, was responsible for the low bid for the project. The total bid price was well below the design engineer's estimate, and below the eight other bidders that based their costs on drill-and-blast construction.

The Beverly Sewer Tunnel TBM. A reconditioned 8.5-foot diameter TBM was chosen to excavate the Beverly Sewer Tunnel. Figures 6 and 7 show generic TBM components for a similar TBM. The principal components are the cutterhead, the main body and the muck removal system. The cutterhead of this full-face, hard rock TBM consists of a number of disc cutters mounted on a circular, flat or convex head (see Figure 7). The cutterhead is rotated by electric motors through a main bearing attached to the main body. The Beverly TBM cutterhead rotated at a speed of 12.5 revolutions per minute. As the cutterhead rotates, thrust cylinders press the disc cutters into the rock mass. The disc cutters (see Figure 8) cut grooves, or kerfs, into the rock mass and rock chips (muck) are formed through shear and tensile failure of the rock (see Figure 9).

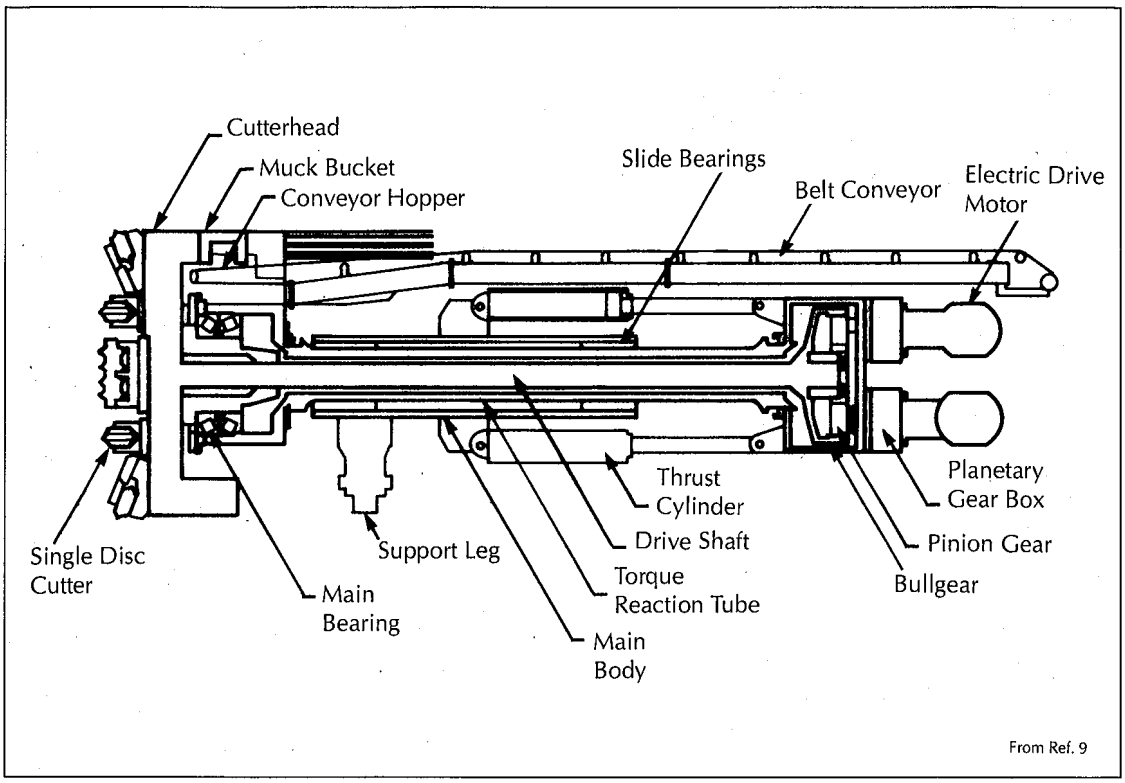

FIGURE 6. Schematic sketch of the TBM components.

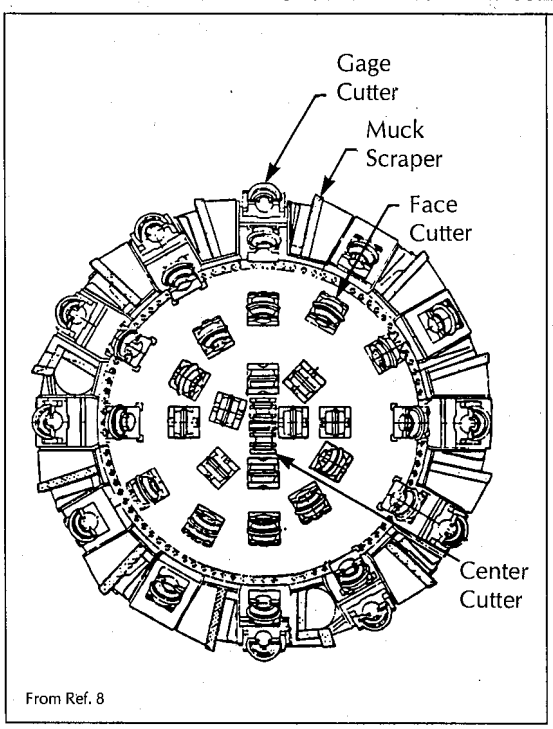

FIGURE 7. Schematic sketch of a TBM cutterhead.

Figure 10 shows the cutterhead of the TBM holing through at the Water Street Shaft. A total of 18 disc cutters, 15.5-inches in diameter, were mounted on this TBM, one more than the normal complement for this type of TBM.

Muck scrapers, the conveyor hopper and the conveyor system constitute the muck removal system. The muck is captured by muck scrapers (see Figure 7) or buckets that pick up muck as the cutterhead turns and deposit the debris onto a conveyor system (see Figure 6) that transports the muck to the rear of the TBM. Once there, the muck falls into rail cars that deliver the debris to the tunnel shaft for final disposition.

The main body structurally supports the cutterhead and mucking systems and houses the torque, steering, stabilizing and thrusting systems that operate the TBM. The torque system on this machine consists of three electric motors that drive the cutterhead. The thrusting system consists of hydraulic cylinders that extend and retract the cutterhead. A number of gripper pads (four on this machine) stabilize

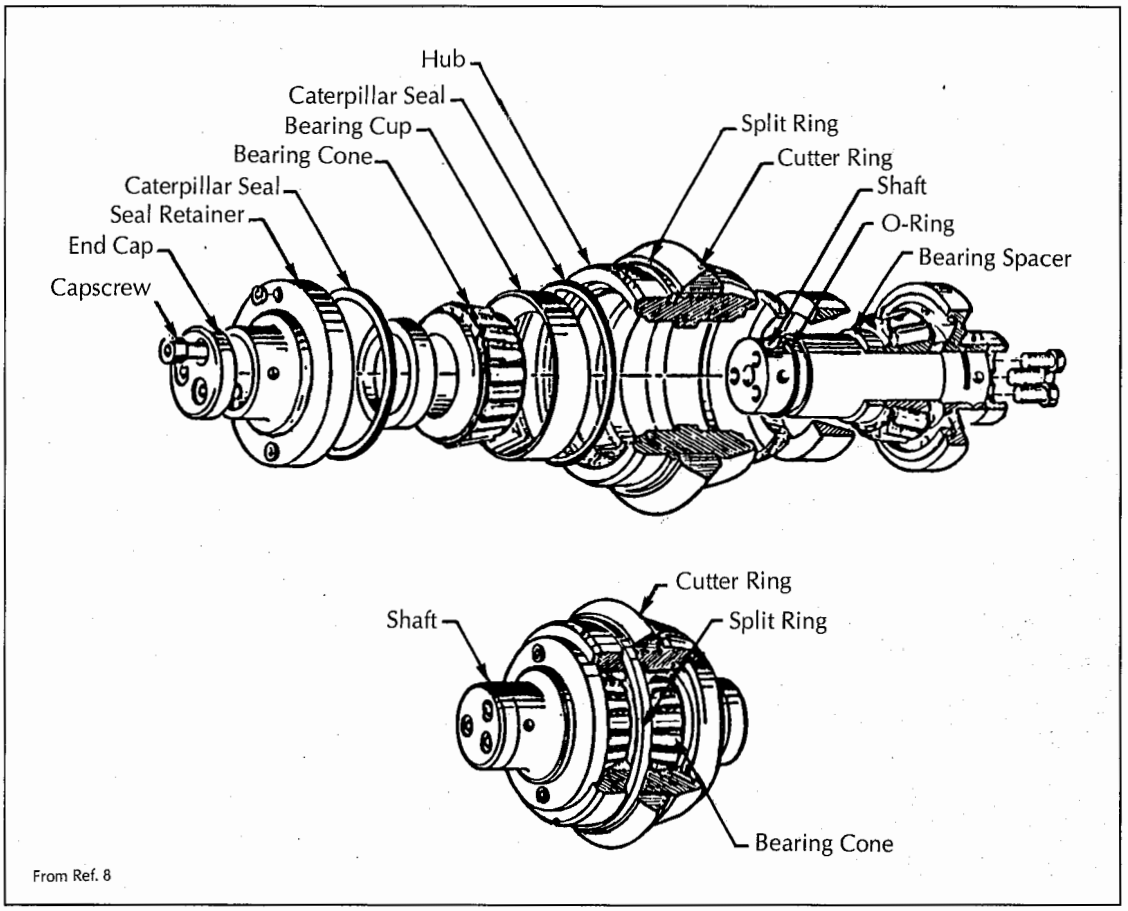

FIGURE 8. Schematic sketch of the disc cutter.

and support the cutterhead, and permit adjustments to the orientation of the TBM so that the appropriate alignment can be maintained. Support legs are used in conjunction with gripper pads to advance and reposition the TBM after each stroke cycle.

This TBM had most recently been operating in less abrasive, softer sedimentary rock, so a number of modifications were made, in addition to installing the extra disc cutter, in order to allow the machine to operate effectively in the hard, igneous rocks in Beverly. The electric drive motors and hydraulic system were upgraded to provide for the greater cutterwheel torque and thrust force needed to negotiate the hard rock environment. Typically, this type of TBM has a total drive horsepower of 375 horsepower (hp), a maximum thrust force of 280 tons and a maximum anchor force of 840 tons.[9] The Beverly TBM was upgraded to

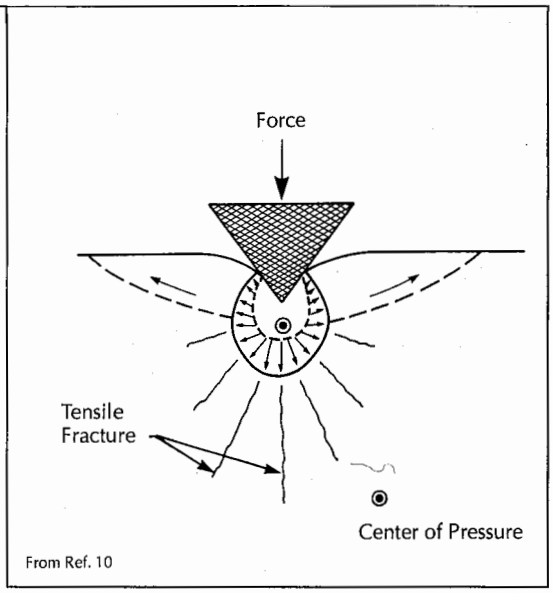

FIGURE 9. Schematic sketch of the failure mechanism for fragmented rock with a TBM.

FIGURE 10. The cutterhead holing through the Water Street Shaft.

a total drive horsepower of 450 (three motors at 150 hp each, 460 volts AC, three-phase, 60 Hertz), a maximum thrust force of 300 tons (33,000 lbs per cutter) and a maximum anchor force of 900 tons.

TBM Performance at the Beverly Sewer Tunnel

The reconditioned TBM commenced mining on April 30, 1990, and "holed through" at the Water Street Shaft on August 4, 1990 (see Figure 10), for an average advance rate of approximately two feet per hour (ft/hr) or about 19.3 feet per day (for an average 9.6-hour day). The highest daily advance was 40 feet in a 10.25-hour day (or 3.9 ft/hr). The average penetration rate (rate of advance when the TBM is cutting rock at the face) was approximately five ft/hr, while the highest penetration rate was about 6.9 ft/hr.

One means of evaluating the performance of a TBM is through machine utilization, expressed as a percentage of the total construction time during which the TBM is in operation mining and mucking rock. Utilization herein includes all operational mining time, including the time required to restroke and realign the TBM before commencing another stroke, or push. Restroking time for this project was estimated to be about eight percent of the machine operating time.

TBM Utilization at the Beverly Sewer Tunnel. Utilization records at the Beverly Sewer Tunnel were maintained by the geotechnical engineer that was present daily during the operation of the TBM. Construction involved a single shift typically of seven to 13 hours duration, six days a week. Operational and non-operational times were recorded in 15-minute increments along with the reason or cause of delay. Average utilization of the TBM at the Beverly Sewer Tunnel project was approximately 42 percent of total

TABLE 5
Summary of the Ten Most Significant Causes for TBM Delays on the Beverly Sewer Tunnel

Description of Delay	Delay Category	Percent of Construction Time
Inspect, Repair & Replace Disc Cutters	TBM Maintenance & Repair	10.0
Awaiting Muck Train*	Back-Up System Maintenance & Repair	8.0
Electrical Problems	TBM Maintenance & Repair	6.1
Repair & Replace Conveyor	TBM Maintenance & Repair	5.6
General Maintenance	TBM Maintenance & Repair	5.0
Dust Guard Repairs	TBM Maintenance & Repairs	4.6
Groundwater Delays	Geology	4.6
Install Trailing	Back-Up System Maintenance & Repair	4.1
Installing Track & Utility Lines	Back-Up System Maintenance & Repair	3.0
Clearing Conveyor Jams	Geology	2.4

* A single track was utilized without switching capability. Locomotive delays started at about 700 feet from the shaft when the TBM could be restroked and reset prior to the locomotive transporting two muck cars to the shaft and then returning with two empty cars.

construction time.

Utilization depends on the ability of the tunneling equipment to operate in a geological environment supported by ancillary and associated back-up systems. Consequently, the factors that result in delays, and consequently influence utilization and performance, are attributable to three primary factors:

- Geological conditions
- Back-up systems
- The TBM itself

Geological conditions "represent the major uncertainty with respect to the prediction of utilization,"[10] since pre-construction subsurface explorations rarely provide complete insight into the geological problems that will be encountered during construction. Typical delays attributable to geological conditions encountered during this project include those resulting from groundwater inflow, supporting or reinforcing the rock mass with steel sets or rock bolts, and those associated with conveyor jams or hand excavating muck from the tunnel invert. Delays resulting from geological factors accounted for eight percent of the construction time.

The maintenance, repair and operation of back-up equipment and systems affect utilization indirectly. Back-up systems delays include those stemming from the installation of utilities (*i.e.*, water, electric, ventilation and compressed air) and/or railroad tracks, the power supply, muck train delays and shaft operations. Also included in this category were delays associated with adding additional segments of the trailing gear once mechanical tunneling had begun. Back-up sys-

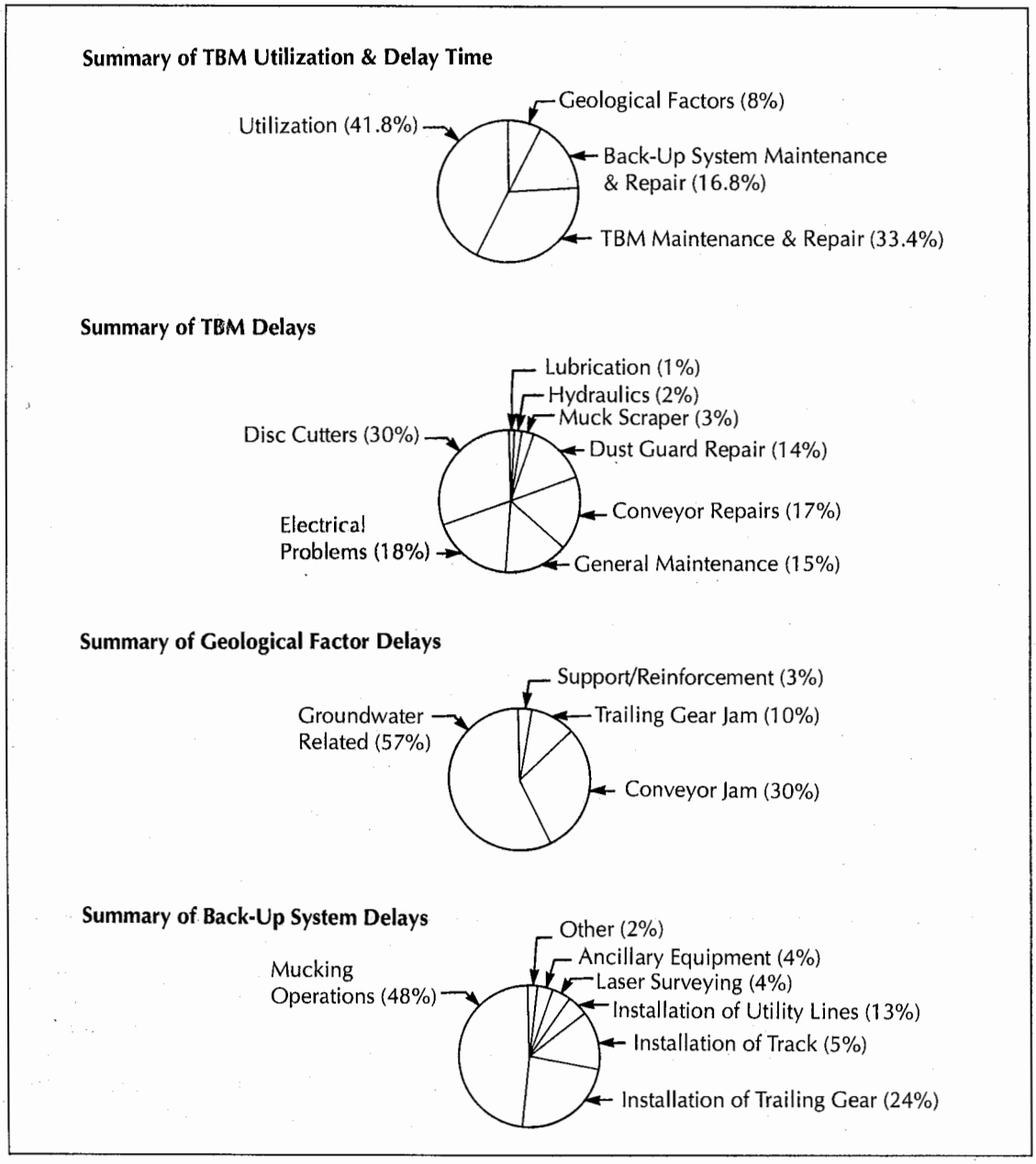

FIGURE 11. Utilization summary for the TBM at the Beverly Sewer Tunnel.

tem delays accounted for 16.8 percent of the total construction time.

The maintenance and repair of the TBM directly affect utilization. For this project, TBM maintenance and repair delays include those associated with general maintenance and inspections, with inspecting and replacing disc cutters, with hydraulic and lubricating systems, with motor and/or electrical problems, and problems associated with muck conveyors. Maintenance and repair of the reconditioned TBM accounted for 33.4 percent of the available construction time.

Table 5 summarizes the ten most time-consuming causes for delays on the Beverly project. Figure 11 illustrates the percentages of utilization and non-utilization time with a breakdown by delay category of the latter.

Benefits of TBM Use on the Project

The use of a TBM on the Beverly Sewer Tunnel Project produced many benefits, specifically those dealing with tunnel support requirements, schedule and disturbance to the community.

Tunnel Support Requirements. A total of 120 rock dowels and six steel sets were installed in order to reinforce and support the rock mass within the tunnel. For the most part, rock wedges in the crown and at the springlines were stable. Rock falls did not occur and those discrete rock wedges that were potentially unstable were reinforced with rock dowels by the contractor.

Steel sets with timber lagging were installed where a series of relatively flat lying, gouge-filled joint sets were encountered and where slaking or raveling of the clayey gouge material was feared. Based on the anticipated support requirements (up to 20 percent of the tunnel drive with pattern bolts) for carefully executed smooth blasting, the estimated number of dowels, including spot dowels, that would have been required would have been over 200. Thus, the non-destructive tunneling process afforded by the TBM reduced anticipated tunnel support requirements by about 40 percent, thereby reducing tunnel support costs, as well as reducing potential rock fall hazards to workers.

Construction Project Schedule. Contract documents required that construction be completed within 540 days subsequent to the formal execution of the contract agreement that occurred on October 18, 1989. Tunneling was completed on August 4, 1990, only 97 days after the start of boring work and 165 days ahead of the design engineer's estimated tunnel completion date, which had been based on conventional drill-and-blast tunneling. Thus, the use of a TBM reduced the duration of tunneling work by over 60 percent compared to the anticipated drill-and-blast duration.

Construction Vibrations. From the early stages of the project, the impact of construction blasting at the shafts and in the tunnel in close proximity to residential housing was a major concern of the SESD. Blasting operations at the Bass River and Water Street shafts were monitored to ensure compliance with the specified ground vibration and air overpressure limits, which were aimed at preventing damage to nearby structures and minimizing human discomfort. Twelve shaft rounds and five tunnel rounds (for the TBM starter tunnel) were detonated at the Bass River Shaft; and eight shaft rounds and one tunnel round were detonated at the Water Street Shaft. Blast vibrations were in all cases within specified peak particle velocity limits.

Although the vibration levels from blasting were below safe levels for cosmetic damage to residential structures, there were still complaints about noise and vibration after many of the blasts. These complaints resulted because the level of perception of humans to ground vibrations is about two orders of magnitude below the safe level for damage to structures. Hendron and Oriard have shown how the level of perception of humans compares to actual damage levels (see Figure 12).[11] Although ground vibration levels from the project were well below "safe" limits, they were in the range of "disturbing" to people. Combined with the air blast overpressures, which would tend to rattle windows, these levels were high enough to result in complaints, especially by people who did not hear the warning whistle and were startled by the noise and vibration. Most of the complaints were for disturbance only, although there were several complaints of alleged damage resulting from the blast vibrations. Subsequent review of these damage complaints indicated that they were not blasting related.

The vibrations from TBM excavation, on the other hand, were less than 0.01 in/sec at the ground surface above the tunnel, which is below the level of perceptibility of humans as shown in Figure 12. Complaints about construction activity ceased once mechanized tunneling began. This abatement highlights one of the advantages to TBM usage — reduced ground vibration and air overpressure levels — and demonstrates the reduced environmental impact of TBM usage on the local community.

Measured Deformations Above Tunnel. Measured ground surface settlements above the tunnel were in all locations less than 0.1 inch, and in most cases were less than 0.05 inch. These small settlements were attributed to:

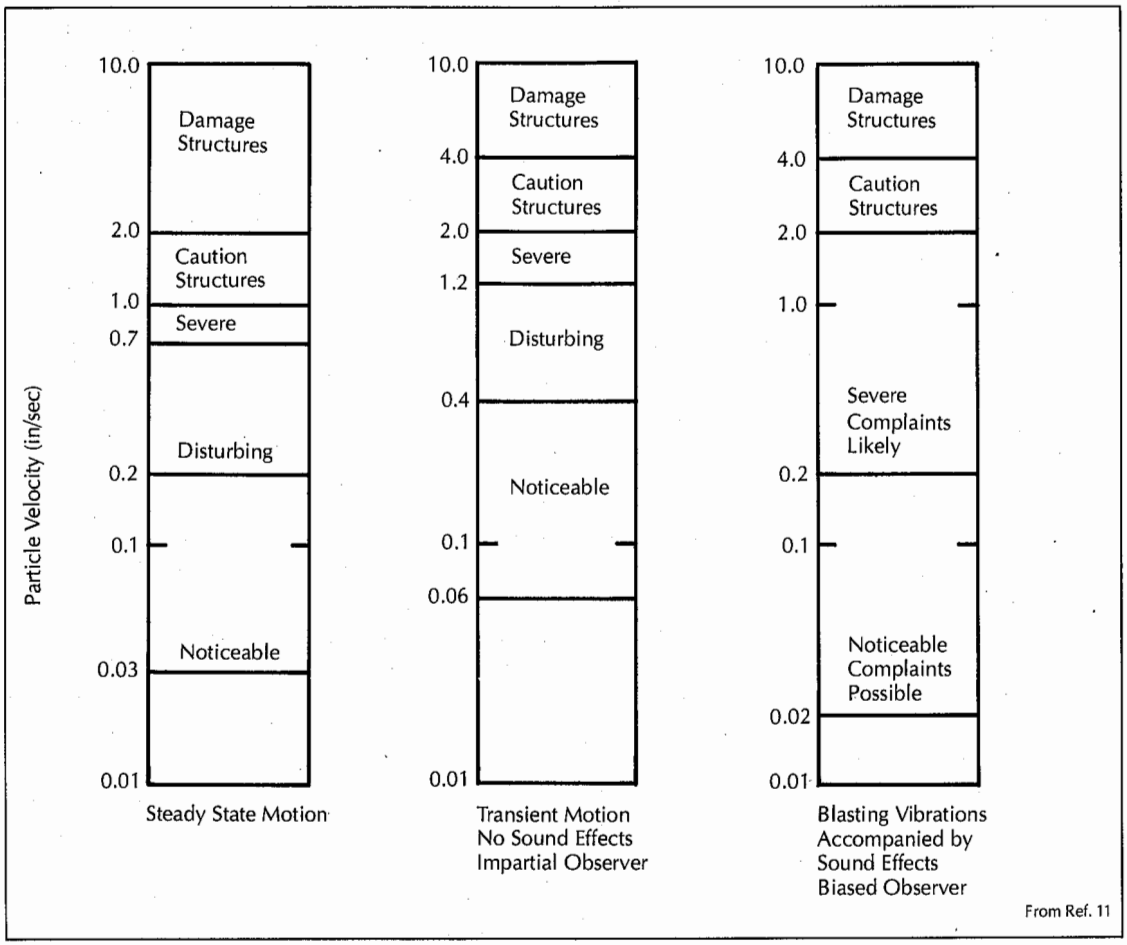

FIGURE 12. Human response to blasting.

- Small deformations in the rock mass above the tunnel (less than 0.05 inch as measured by MPBXs).
- Very small consolidation related settlements in marine and glacial soils above the tunnel that resulted from groundwater drawdown.

Deformations above the tunnel would also have been anticipated to be small with drill-and-blast construction. The major concern for deformations was for possible loss of ground into the tunnel in areas near the Bass River Shaft where there was minimum rock cover over the tunnel crown. With drill-and-blast construction, the potential for such loss of ground would have been greater due to the possibility of explosive gasses opening joints or causing overbreak above the crown.

Summary & Conclusions

A reconditioned TBM successfully completed approximately 1,600 feet of a 8.5-foot diameter tunnel in a hard, high-strength igneous bedrock in Beverly, Massachusetts, with an average advance rate of 2 ft/hr, or 19.3 ft/day, for an average 9.6-hour day. The average penetration rate was about 5 ft/hr.

Comparing the TBM method to drill-and-blast tunneling for this project, the TBM excavation method resulted in:

- A 40 percent reduction in tunnel support requirements over those which were estimated for careful smoothwall blasting.
- A 60 percent reduction in tunneling time over that which was estimated for drill-and-blast tunneling.

- The elimination of complaints from residents about noise and vibration.

The utilization rate (including restroking) of the TBM was approximately 42 percent of total construction time. Delays due to geological conditions accounted for eight percent of construction time. Delays for maintenance and repair of the TBM accounted for about 33 percent of construction time. Delays for maintenance and repair of back-up systems accounted for about 17 percent of construction time.

The performance and utilization of the reconditioned TBM on the Beverly Sewer Tunnel confirm that TBMs can operate effectively in hard, high-strength igneous rocks native to Massachusetts. Furthermore, short tunnels, which historically have been bid as conventional drill-and-blast tunneling operations, can be mined with reconditioned TBMs that are available at substantially less cost than a new TBM and that can be mobilized more rapidly.

While public response to the mechanical excavation was not measured directly, it should be noted that during shafting and starter tunnel operations involving drill-and-blast techniques, public complaints were directed to the contractor, the police department and/or the fire department after every blast round. Complaints were generally about disturbance, although several complaints were made regarding damage (which were not substantiated). By contrast, no complaints were raised during the TBM tunneling operations. Therefore, the use of a TBM in tunneling in the vicinity of residential or commercial neighborhoods has significant advantages over drill-and-blast tunneling from an occupant disturbance and public relations standpoint, as well as in limiting the potential for vibration damage or damage claims.

In consideration of these points, tunnel designers should make provisions in contract documents for the use of TBMs (even in short, hard rock tunnels), if the tunnel has a commonly used diameter or if there is any flexibility in tunnel size.

ACKNOWLEDGMENTS — *The TBM employed for this project was a Jarva Mark 8. The owner of the project was the South Essex Sewage District (SESD). Design Engineers were Metcalf & Eddy, Inc., of Wakefield, Massachusetts. Haley & Aldrich, Inc., of Cambridge, Massachusetts, served as geotechnical and tunneling consultant on the project. The general contractor was Mole/Kassouf Joint Venture of Solon, Ohio. The authors wish to acknowledge James Ryan, Project Manager from Metcalf & Eddy, and Raymond Bouchard, Project Engineer for the South Essex Sewage District, for their support and assistance during design; and Robert M. Birch and Robert Curry of the Mole/Kassouf Joint Venture, and Richard Whalen, resident engineer for Metcalf & Eddy, Inc., for their cooperation and assistance during construction. They also wish to thank Bruce Beverly, James Smith and Daniel Dobbels, at Haley & Aldrich, for their comments and suggestions regarding this article; and Acey Welch and Terry McEleney for their assistance in preparing the figures.*

GEORGE W. HARTNELL, III, *is a Staff Engineer at Haley & Aldrich, Inc. He served as the resident engineer during shaft and tunnel construction on the Beverly Sewer Tunnel project. He received his B.S. from the United States Military Academy at West Point in 1979 and an M.S.C.E. from the University of Illinois in 1986. His underground experience includes providing technical expertise to the Defense Nuclear Agency on soft ground and hard rock tunneling operations, and providing design recommendations and field monitoring for several underground structures in soil and rock. He is a registered professional engineer in Virginia.*

ANDREW F. MCKOWN *is a Senior Specialist with Haley & Aldrich, Inc. He served as project manager for the Beverly Sewer Tunnel project. He received a B.S. in civil engineering from Tufts University in 1969 and an M.S.C.E. from the Massachusetts Institute of Technology in 1978. His experience includes a wide variety of geotechnical projects, including several hard rock, soft ground and mixed face tunnel projects, and many projects involving deep cuts in soil and rock. He is a member of BSCE, the ASCE Rock Mechanics Committee, the International Society for Rock Mechanics, the Underground Technology Research Council and the Society of Explosives Engineers. He is a registered professional engineer in Massachusetts.*

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Towards Formulating an Ethical Transportation System

Changes in transportation policy, as well as how that policy is perceived, require a transportation system that utilizes intermodalism & new technologies & that offers choice.

GIL CARMICHAEL

Changes are taking place in transportation policy at all levels of government. These changes challenge current thinking about transportation, providing a way to rise above the thinking that results from the pressures and constraints of day to day work. No matter what the predisposition to transportation policy, it is not difficult to realize the very basic and important fact that our nation's policies toward transportation are in flux. The main uncertainties seem to revolve around the pace of this change and the full dimensions it may take along the way.

Catalysts for Change

Congestion is a catalyst for change. The congestion issue has snowballed in significance during the last five years. Highway congestion was the driving force behind the voter initiatives in California that earmarked billions of dollars for transportation improvements for not only highways, but also for mass transit and railroads. Airport congestion prompted recent action by Congress that encompassed a variety of aviation access, funding and noise control issues. Congestion is the triggering point for leaders of the Northeastern states who are pondering the most efficient way to move travelers between Boston and New York. Congestion affects both our nation's passenger and freight transit systems.

The environment is a catalyst for change. Opinion polls consistently show a strong reservoir of public support for environmental initiatives. Congress has just enacted sweeping revisions to the Clean Air Act. While it is true that California voters — a state that many consider to be at the forefront of environmental activity — have turned down the so-called "Big Green" environmental initiative, the MacDonald's Corporation has announced that it will abandon plastics-based packaging for paper packaging as a pollution-reduction measure. Which is the more potent national trendsetter — California or the folks who operate under the golden arches? The nation's interest in a cleaner environment most definitely affects choices for both passenger and freight transportation pol-

icy, since virtually every transport hub in this country lies in a region of inferior air quality — in what the federal Environmental Protection Agency's (EPA) terms are "non-attainment" areas.

Money, or the lack of it, is a catalyst for change, too. The deferred maintenance on our nation's overworked highway system is at unacceptable levels, forcing highway officials to concentrate on repairing existing roadways rather than undertaking new projects.

One of the main themes that emerged from the U.S. Department of Transportation's National Transportation Policy was the realization that there will not be enough taxpayer or user fee dollars available to fund both the maintenance of existing highways and the creation of new ones. In many cities, the cost of new airports is prohibitive, even if acceptable sites could be found. Greater reliance on private sector financing of transportation improvements is a necessity.

The emergence of a global economy is changing the way transportation issues are viewed. A global economy outlook especially affects how planners approach maintaining and improving the nation's freight system. Since the cost of transportation is an important component of the cost of goods, its cost must not rise too high if our nation is to be competitive in a world marketplace. The emergence of an economically united Europe has forced our government to reexamine the potential of the three national economies of North America. Changes in the marketplace are beginning to suggest the vast potential for trade among the U.S., Canada and Mexico. Indeed, the auto parts industry already illustrates how this intracontinental trade could work. Expanded railroad traffic in manufactured goods, or in paper for recycling, or in Platte Valley corn is moving through the border gateways. These international operations now involve all three nations on this continent.

Another catalyst for change in transportation is what people in the industry call *intermodalism.* Simply put, intermodalism is the efficient tying together of more than one mode of transportation in order to create greater efficiencies in the movement of people or goods. Unit trains carrying double-stack containers that originate from ocean liners in California or Puget Sound to midwestern destinations are but one example. However, the record of applying intermodalism has not been adequate, especially in the movement of people. Dulles Airport, in the remote suburbs of Washington, D.C., has been in operation for twenty years. Only now are transportation planners beginning to wrestle with such problems as how to move people between Dulles and their homes or offices. They are only now beginning to realize that the trip does not end at the baggage claim. An emphasis on intermodalism — freight and passenger — may be the surest way to increase the efficiency of our nation's transportation system and prepare it for the demands of the 21st century.

The final catalyst for change is the growing recognition on the part of transportation professionals, engineers, and government leaders and officials at the local, state and federal levels that our country cannot build its way out of gridlock. The nation cannot afford it and public sentiment clearly indicates that there is little support for such a solution. Increasingly, the public and local governments are saying that the future does not rest in paving over the landscape with new highway lanes or airport runways. Local and state officials are telling the federal government that they want the flexibility to spend transportation dollars in the way that makes most sense in their home areas. That message is being heard loudly and clearly in Washington.

Acting on Change

Change is coming. How do we take advantage of it? First, it is imperative to recognize that all of the aforementioned catalysts, plus others such as safety and the cost of fuel, are tied together. Any solution of one problem area must be viewed in terms of how it will affect solving others. The best solution to any one problem area is that which has the best overall effect. Our nation's railroad system offers a useful option in resolving several of these problems, providing a promising future.

Regarding congestion, railroads have the corridors with excess capacity, both freight and passenger, into the heart of the most congested urban regions. Regarding pollution, railroads

move people and goods with lower levels of emissions per person or per ton of freight than highway vehicles. Regarding money, the nation's freight railroads operate in the private sector, unlike the beleaguered highway system. These railroads annually spend approximately $3 billion in capital improvements — money that does not come from taxpayers. The main line rail system of this country is in the best condition in its history. Since the deregulation of the industry a decade ago, hundreds of new local and regional railroads have emerged to serve rural areas and the important national network of small- and medium-sized cities.

The benefits of railroads underscore the fact that there are answers to the nation's transportation needs. Each mode — aviation, waterways, rail, truck, inland barge or pipeline — has functions it performs best in given settings. The key to the equation is finding the strength of each mode and linking each by that strength.

However, in order for each mode to find its niche and create a strong interconnected system, it is important that government policies do not distort this process of evaluation and connection. One of the objectives of the U.S. Department of Transportation's National Transportation Policy is to promote equity among all of the different modes, to create a "level playing field." For example, the freight railroads will not be able to make use of their excess capacity if government policies either subsidize their competition or impose unique economic burdens on them. Just about every credible study suggests that these inequities still persist.

The Role of Intermodalism

Whether seacoast port improvements or better links between airports, downtowns or other transportation systems are planned, there is an overwhelming need to look at the benefits of tying all of these modes together. In most regions of the country, short-distance commercial airline flights have become expensive, because it is so costly to operate the current generation of trunk airline equipment over these routes. Passenger rail service via Amtrak between New York and Washington now represents the market share leader in that region, overtaking the air shuttle. The New England states are working with the federal Department of Transportation to determine if high-speed surface transportation can move people more conveniently between Boston and New York, thus freeing up additional much-needed airport capacity for the more profitable long-distance flights.

Texas is moving forward with its ambitious high-speed rail project. Other regions of the nation are planning for the high-speed surface transportation requirements of the 21st century — California, Florida, Michigan, Nevada, New Mexico, the Northeastern states, Ohio and Pennsylvania. These states are contemplating utilizing three types of projects that will implement new technologies — magnetic levitation trains, high-speed steel-wheel-on-steel-rail trains, and tilt trains — in addition to expansion of conventional passenger rail service via Amtrak. These three new technologies are important to the development of surface transportation in the United States in the next century.

As each of these advanced technology systems begins to take shape and is used, it will bring benefits to where it is applied. These advantages will take the form of improved service in the city pairs affected, better connections to the national transportation system (aviation, rail and transit) and more choice for the consumer. Choice is important both for price and convenience. Choice stimulates competition. Choice provides an alternative. Even confirmed air shuttle riders between New York and Washington flock to Amtrak on snowy or foggy days.

Summary

The particular analysis of transportation issues presented herein foresees a stronger role for our nation's railroads based on the development of what may be called an "ethical" transportation system. The word, *ethical,* may seem, perhaps, a strange word to apply to something as commonplace, and to something that would appear to be far removed from the moral sphere, as transportation. But it is an apt term, given the new national attention to ethical behavior. However, what would be an "ethical" transportation system? An ethical transportation system does not kill or harm people. It does not waste energy. It does not pollute the air, water or land. An ethical transportation system, by

nature, is a system that is built on sound economics, since if a service flunks the basic tests of economics, it would be difficult to rank it very highly in other terms.

As the leaders and transportation professionals of our nation ponder our transportation options, railroads come out very favorably on the ethical criteria. Their safety record is excellent and improving. Their energy efficiency is better and, therefore, their emissions are significantly lower than other petroleum-based land transportation (highways and aviation). The excess capacity of the existing rail system reduces the need to acquire land for new routes as has been done for the highway system for decades. Railroads — freight and passenger, conventional and high tech — fit into the future of the United States. Study of the current various opportunities and projects mentioned here, as well as many more in this country and in other parts of the world, is essential in developing, implementing and maintaining a workable and ethical transportation policy.

NOTE — *This article is based on a speech delivered to the Boston Society of Civil Engineers Section/ASCE at the F.M. Keville Memorial Dinner on April 24, 1991.*

GIL CARMICHAEL *was appointed Federal Railroad Administrator (FRA) in September 1989. He has supervised the implementation of the Rail Safety Improvement Act, which conferred new safety enforcement authority on the FRA. Under his leadership, the agency's safety inspector workforce has been expanded and a new training program is being developed. He was appointed by President Ford as a Federal Commissioner for the National Transportation Policy Study Commission and served as Chairman of its Subcommittee on Advanced Technology. From 1973 to 1976 he served on the Department of Transportation's National Highway Safety Advisory Committee and was its chairman in 1975 and 1976. A graduate of Texas A&M University with a B.S. in business, he was a fellow of the Institute of Politics, Kennedy School of Government, Harvard University. From 1980 to 1989 he was a trustee of the Robert A. Taft Institute of Government.*

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