CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

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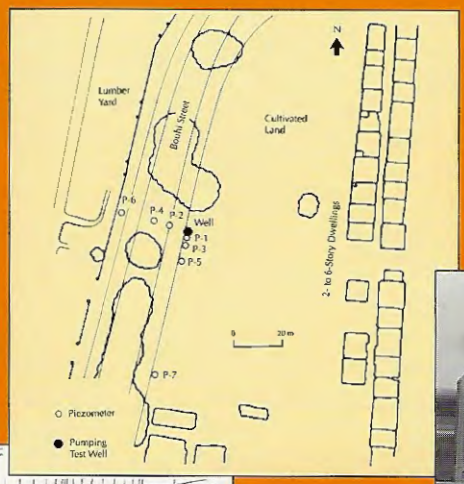
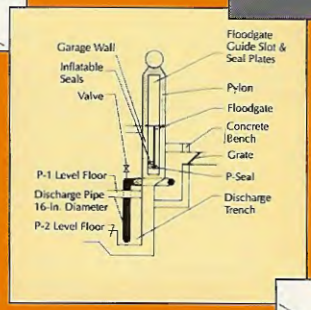
Deep Well Dewatering in Egypt

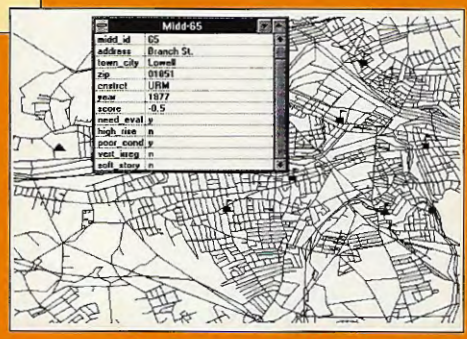


Electronic Traffic Management in Italy

Construction Simulation Software



Riverside Flood Protection



Reducing Seismic Risk

Also in This Issue:

- Design of a Confined-Site Highway Ramp
- Suspension Bridge Anchorage System Design

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Editorial, Circulation & Sales Office:

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

An Innovative Design for the Flood Protection System of a Riverside Development

The use of steel barrier gates raised by cranes, counter-flooding and an early warning system combine to provide reliable and cost-effective flood protection.

GUNARS RICHTERS

Located on approximately 3.5 acres along the shores of the Potomac River, Washington Harbour is a development with shops, restaurants, an office building and condominium complex, and a 580-car underground parking garage — all sharing a common foundation (see Figure 1).

One of the most innovative features of the development's design, and vital to its success, is virtually invisible to the casual observer. There are 50 floodgates hidden in vertical pockets beneath Washington Harbour's plaza. These retractable, watertight floodgates protect its people and property without obstructing its scenic view or public access to the river. During major floods, a counter-flooding system balances the hydrostatic uplift that accompanies the rising water.

Project Requirements

Even during the planning and approval process for the development, flood protection was a primary concern. A well-organized opposition, intent on preserving the site as a waterfront park, made flood protection issues the cornerstone of their objections to the development. They argued that it would be effectively impossible to isolate the development, and the people it was intended to attract, from a rising Potomac.

In addition, a geotechnical investigation of the site revealed an aquifer running to the Potomac beneath the proposed development. In the event of a major flood, the hydrostatic uplift forces that would develop would exceed the dead weight of Washington Harbour's structures and would, in effect, cause the complex to float.

As a result, the developers were required to perform an extensive hydraulic study that en-

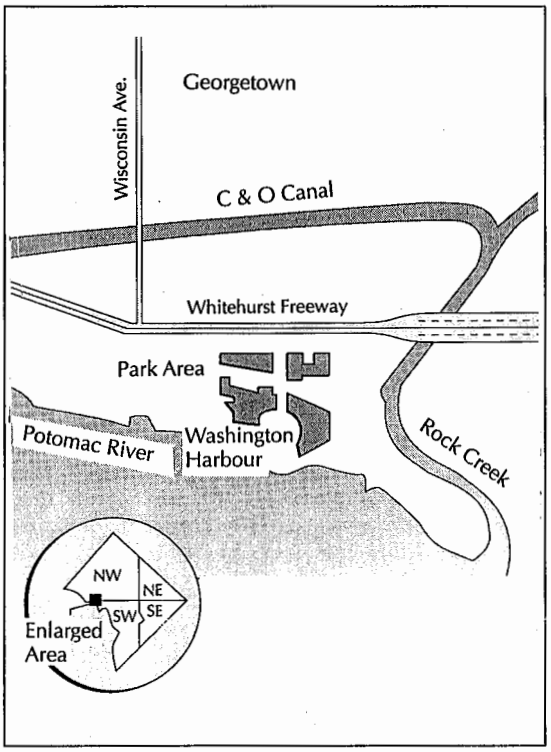

FIGURE 1. Location of the development.

compassed the following:

- Established the 100-year flood level;
- Devised measures to protect the complex and its occupants against a 100-year flood and determined the types of flood barriers and their location;
- Developed a system to neutralize the effects of hydrostatic uplift; and,
- Designed a flood warning system that would provide enough time to make the flood barriers operational.

Study Phase

Working with the civil engineering consultants for the development and Dr. Ron Steinberg of the University of Maryland and using a combination of riverine flow and tidal surge data, consulting engineers for the project determined the elevation of a 100-year flood to be 17.25 feet (MSL). This conclusion was relayed to the Federal Emergency Management Administration. Based on information provided by river gages upstream of the development, this study to establish the 100-year flood level also revealed that, given the configuration of the Potomac River basin and the location of the development, there would be a warning of at least eleven hours of an impending flood.

Flood Barrier Design

Washington Harbour's design called for its retail facilities to be located at the plaza level. Taking this siting into consideration, the engineers worked with the development's architects to define areas where elements of the building could be sited above the 17.25-foot flood elevation. Doing so permitted the building wall to serve as its own flood barrier and ensured that the public would have immediate access to the water — there would be no intervening barriers between the building and the river.

During periods of high water, the complex becomes an island. As Figure 2 illustrates, flood barriers were required for such access points as Virginia Avenue, the entrance to the parking garage, service entrances, and for the decorative windows and main entrance fronting "K" Street.

Several alternatives were considered during the process of developing the design approach to the flood barriers. These alternatives included steel barrier gates, mechanized lift gates, swing gates, inflatable rubber dams, and such conventional measures as stop logs and sand bags. Steel barrier gates proved to be the best alternative since they offered a number of advantages:

- They could be unobtrusively stored in vertical pockets immediately below the surface and encased in a decorative cover;
- They could be sited at the precise location in which they would be needed in the event of a emergency;
- Their operation would be simple and reliable — unlike the horizontally-stored mechanized gates that were also considered but rejected as too complex; and,
- Comparatively little manpower would be required to get them in position.

Adapting technology from nuclear containment plants and working with a floodgate manufacturer, the engineers developed an in-

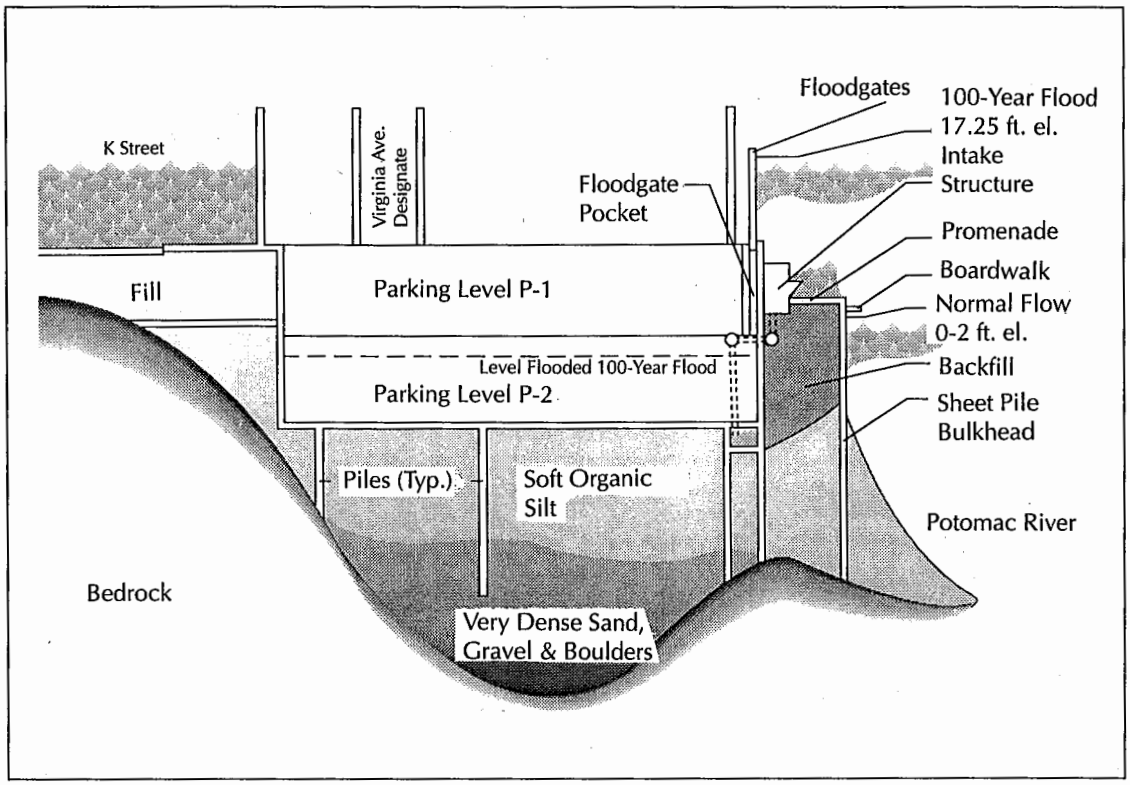

FIGURE 2. Cross section of the development's flood protection system.

novative design solution for Washington Harbour's floodgates. The prototype gate is equipped with two inflatable seals, each designed to withstand the pressures of a 100-year flood. Running continuously around the edges of the gate, these seals provide a watertight barrier when the gate is in its raised position and the seals are inflated. The force of the water on a third P-seal on the back side of the gate effectively ensures that no water whatsoever can get through the floodgate to reach the land side of the development. The water presses the gasket against a steel plate. The steel seal locking device for the floodgate is shown in Figure 3.

For safety reasons, the gates were designed to be locked into place at full height (see Figure 4). The gates slide on channels hidden in decorative pylons, similar to casement windows. Construction of the channels and pressure plates must be exact since the tolerances are small.

Building walls serve as a flood barrier wherever possible (as Figure 5 shows). The walls and the floodgates together provide a 960-foot floodwall around Washington Harbour. In one area, a gate runs continuously for 590 feet. Ranging from four to 26 feet wide and four to 12 feet high, the floodgates are some of the largest and heaviest ever installed anywhere.

To preserve the architectural aesthetics of the development, the gates are hidden between concrete pylons disguised as ornamental columns. When a flood warning is issued, a crane lifts the gates from their concealed pockets, sliding them up within channels in the pylons. Once the gates are raised, their seals are inflated by a portable compressor.

Counteracting Hydrostatic Uplift

At six feet above the flood stage, the hydrostatic pressure created by the rising water is enough to float the dead weight of the Washington Harbour development. Several alternative methods of counteracting this condition were considered.

Slurry Wall. The first alternative considered was to reduce groundwater intrusion by con-

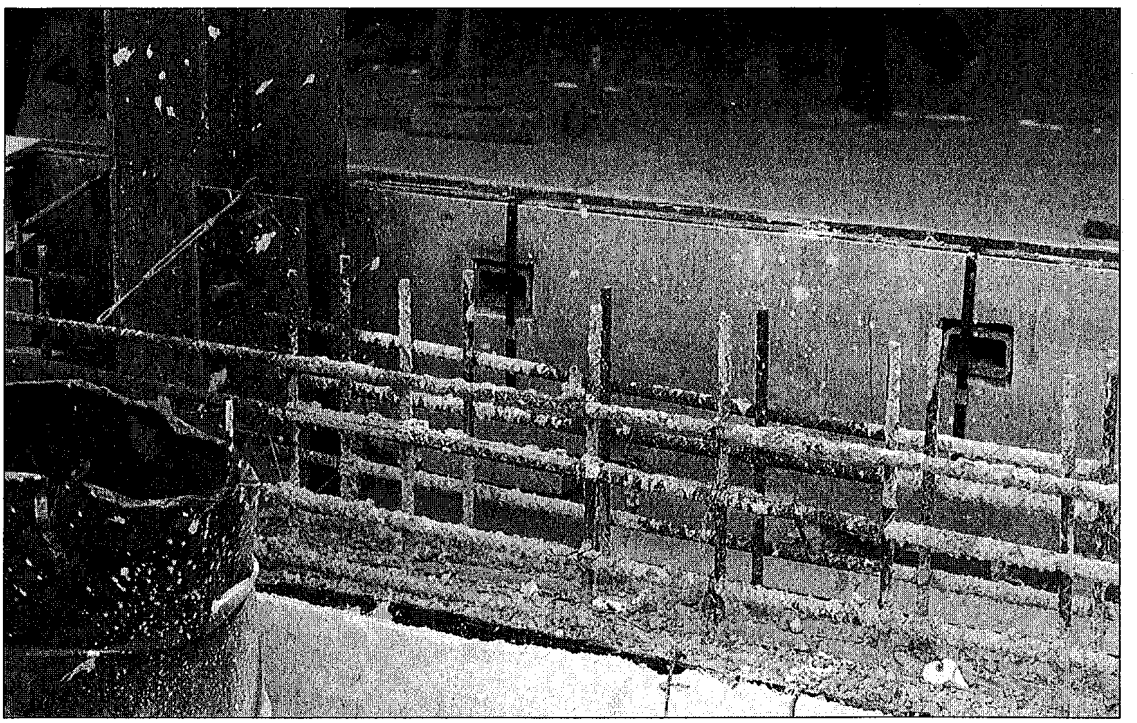

FIGURE 3. A view of the steel seal locking device for the floodgate.

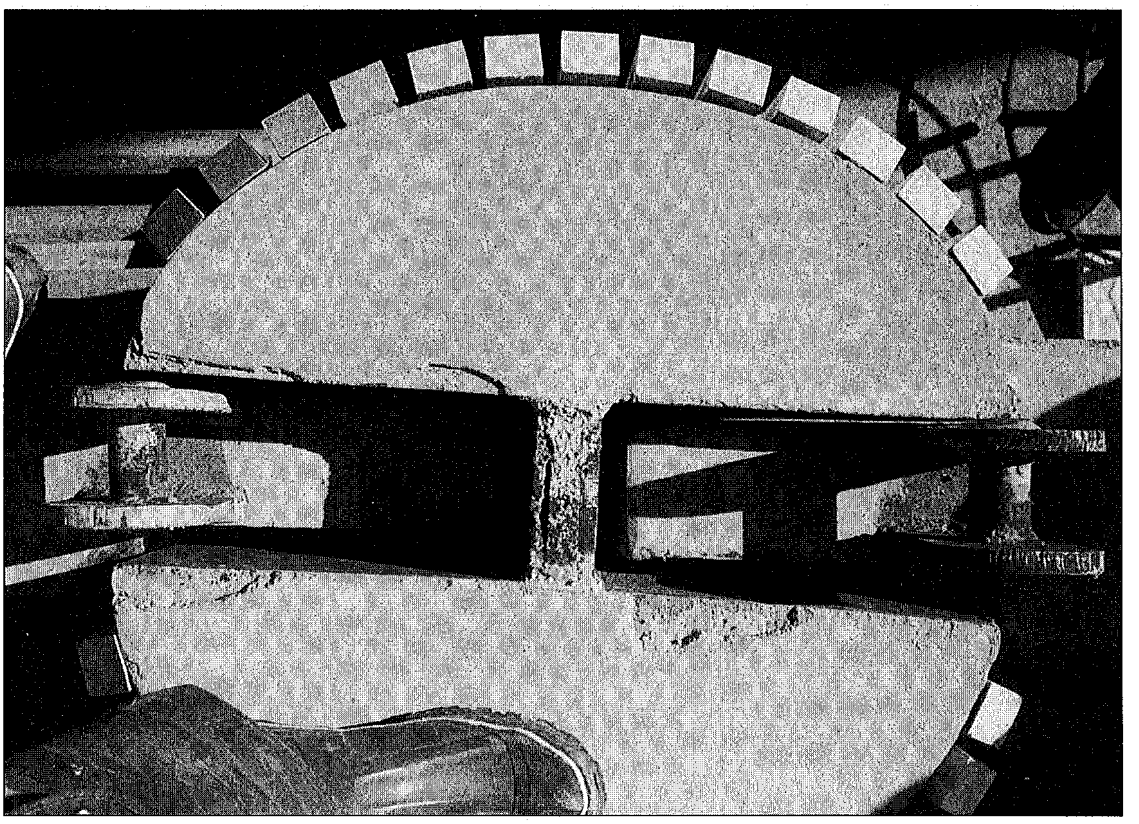

FIGURE 4. A view of the floodgate pylon with the gate in locked position.

structing a slurry wall surrounding the development to isolate it from the river. It was estimated, however, that this alternative would escalate the project budget beyond acceptable limits and delay the schedule by at least nine months since no other work could begin until the slurry wall was in place.

Tie Downs. The use of tie downs, anchored and tensioned in the bedrock, in combination with piles was also considered. This approach was rejected because it was discovered that a "bedrock canyon" ran underneath the site. This feature increased the number of piles that would be needed to resist tie down tension. Moreover, maintaining uniform tension during major floods in an area where rock depths varied from 25 feet to more than 90 feet presented very significant problems.

Relief Wells and Subdrains. A third approach to neutralizing hydrostatic pressure under the development consisted of installing a series of two-foot diameter wells with 12-inch diameter casings and ten feet of well screen designed and constructed to accommodate groundwater flows generated by a 100-year flood. These wells were to be connected to five sump pumps constructed in the lower basement level of the development: three on the river side, and one at each corner on the "K" Street side. As groundwater rose, it would be concentrated at the wells, drained to the sumps and pumped off. While the system would have been reliable, it would have demanded taking steps to mitigate settlement that could have occurred adjacent to the building.

Counter-flooding. The most cost-effective method of neutralizing hydrostatic pressure proved to be a carefully controlled system for counter-flooding the lower level of the parking garage. This system provides built-in redundancy and a safety factor of two in overall systems. Redundancy is created by using three intake structures instead of just one large structure and by using two 16-inch diameter pipes instead of one 20-inch pipe leading from the intake structures to the P-1 parking level. Control valves on these two 16-inch pipes can be operated either manually or by electric motor and are located for easy access. The piping system leads also to the P-2, or lower garage, parking level and discharges into a trench to

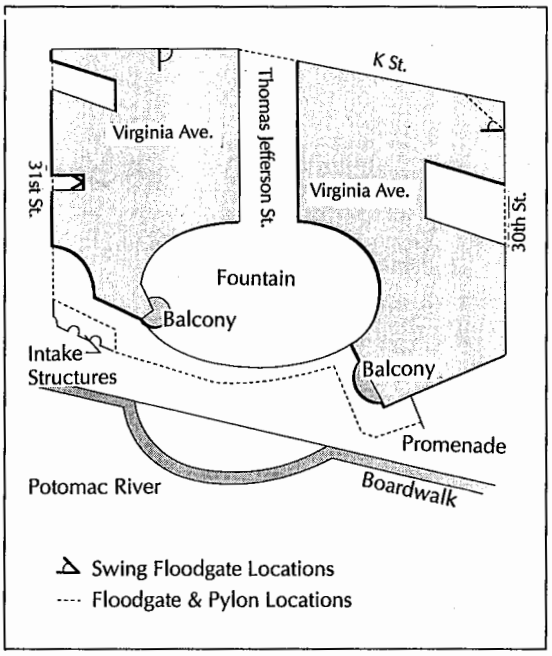

FIGURE 5. Location of building walls and floodgates.

dissipate water energy in a controlled flow.

As Figure 6 shows, the intake structures of the counter-flooding system are located below benches outside the floodgates surrounding Washington Harbour. To prevent debris from entering the garage, the intake structures take water from two to four feet below the debris-filled surface of the flood waters. Inflow velocity is low to prevent debris being sucked into the intakes. Standard two-inch catch basin screens help filter out any debris that is caught. The screens are also slanted to allow debris to drop off. Figure 7 shows a detail of the intake structure.

The thickness of the bottom slab of the lower parking garage was sized so that there would be enough dead weight to resist the hydrostatic pressure of flood water rising to the 10-foot elevation. Above the 10-foot elevation, ballast water is introduced into the the lower parking garage in quantities that correspond to the rise of the river. No more water is introduced than is required to balance the flood level of the river. For the last 6.9 feet above the 10.35-foot elevation (up to the 17.25-foot 100-year flood elevation), an inch of water is introduced into the garage for every inch that the river rises.

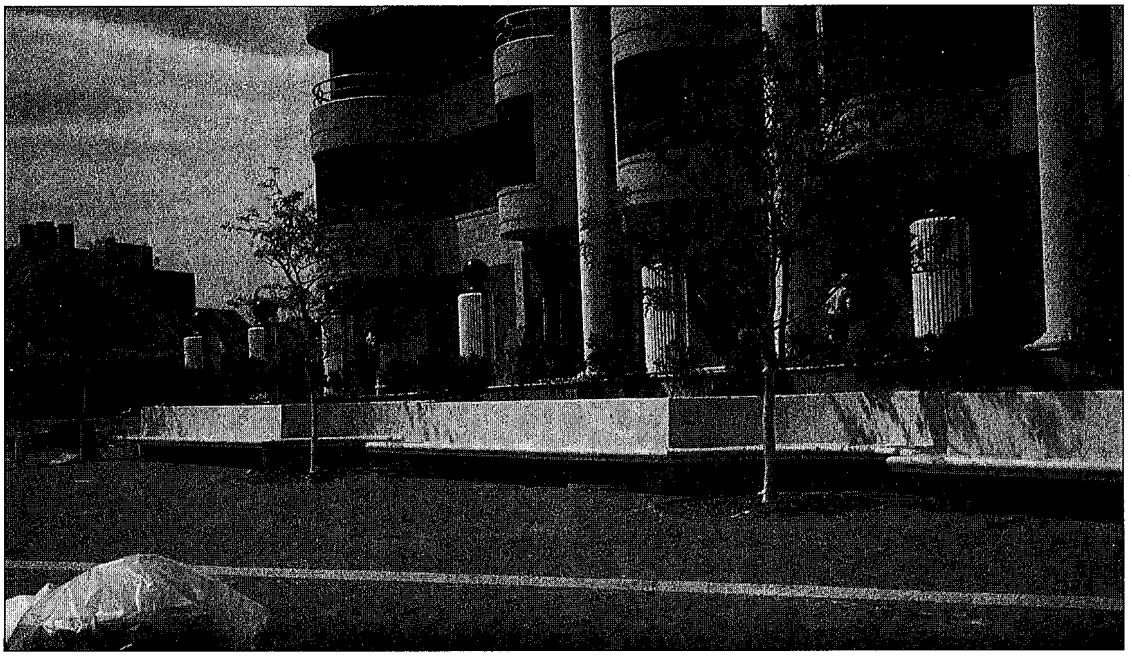

FIGURE 6. A view of the intake structures.

The water ballasting, combined with the 10-foot resistance provided by the oversized bottom slab, yields the 16.9 feet required to resist the hydrostatic pressure of a 100-year flood.

Once the floodwaters have receded, the inundated parking levels are pumped out using two 800-gallons-per-minute pumps located in the southwest corner of the complex. After a 100-year level flood, it will take approximately three and a half days to evacuate the eight million gallons of water needed to offset hydrostatic pressure from the garage.

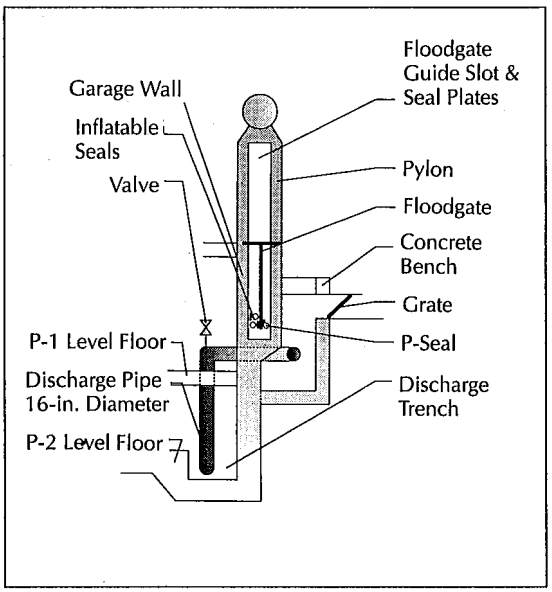

FIGURE 7. Intake structure detail.

Flood Warning System and Operation

Flood Warning System. For Washington Harbour's flood protection barrier to be effective, systems have to be in place in order to provide adequate warning of an impending flood. The National Weather Service (NWS) tracks all storms, predicting their intensity several days before they reach the area. The larger the storm, the longer it is tracked. The NWS also gathers data from numerous gages along the Potomac. These data, along with predicted rainfall within the drainage basin, are used to generate a flood prediction computer model. Warnings are issued to property owners as soon as the computer model indicates that flood water will exceed elevations at given sites.

The effectiveness of the NWS's flood prediction and warning system was demonstrated four times during the construction of the Washington Harbour development. On each occasion the early warning gave the contractor the

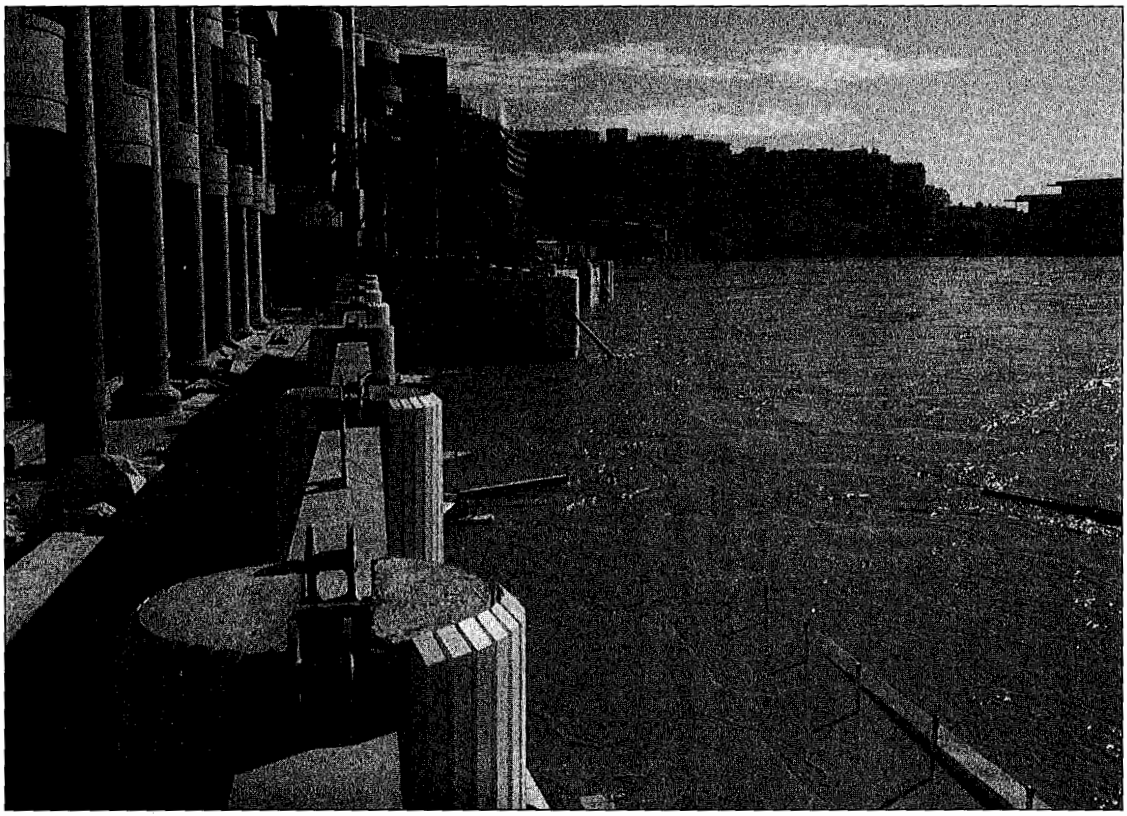

FIGURE 8. A view of the floodgates in operation.

lead time needed to prepare for the floods (Figure 8 shows the barriers in place during one of these instances).

On one occasion, when floodwaters exceeded the 12-foot elevation, the contractor received a warning that gave sufficient time to contact the Washington, D.C., electric company in order to request that Washington Harbour's power supply be isolated from the rest of the city's electric system. Doing so usually takes 24 hours. That the NWS was able to provide flood alerts so far in advance of the event is a testament to the accuracy and effectiveness of the warning system.

Warning System Design and Operation. Based on NWS data, it was established that a six-hour warning would be issued for small tidal surges while riverine flooding would be preceded by a minimum eleven-hour warning. A variety of alternative methods for lifting the flood barriers, inflating their seals, and locking them in place within these times were considered. They included hydraulic systems, permanently installed chain hoists and come-alongs designed to complement the marine theme of the plaza, and crane/fork lift systems.

Hydraulic systems were rejected because their reliability would be adversely impacted by the infrequency of their use. Chain hoists were deemed to be too laborious and too slow to get all floodgates in position in time. Moreover, it was doubtful that chain hoists could keep the gates level as they were being raised to avoid wedging or that, in the event that they did become wedged, they could break them free.

The developers ultimately decided that the most effective way of putting the floodgates in place was to raise them with a crane. Experienced crane operators and riggers can center the hoist to minimize the chances of wedging the gates in their slots while a crane provides sufficient power to dislodge a gate if it does become wedged.

The decision to raise the gates by crane has proved to be a wise one. The crane, which must

be stored off site, can be mobilized and then all the gates around Washington Harbour raised within a couple of hours — well within the warning window provided by the NWS.

GUNARS RICHTERS *is a Geotechnical Engineer with the Sverdrup Corporation in Boston. He joined Sverdrup in 1968 and holds a B.A.S.C. in Civil Engineering from the University of Waterloo in Canada. He is expert in the planning, supervision and analysis of subsurface investigations; in geotechnical analyses; and in the preparation of geotechnical state-of-the-art review. He has experience in a variety of areas, including building/construction, transportation and hazardous waste remediation/assessment. He co-authored "Cut-and-Cover Tunneling Techniques: A Study of the State-of-the-Art" for the Federal Highway Administration's Office of Research in 1973.*

Deep Well Dewatering for the Greater Cairo Wastewater Project

Pumping test deep well results can be applied to design modeling, provided the test represents the actual system and is performed at comparable pumping rates.

ROBIN B. DILL & MARK M. PETERSEN

An extensive confined aquifer system up to 90 meters in saturated thickness lies beneath the greater Cairo, Egypt, area. This aquifer is recharged mainly by the Nile River.

Construction of the Greater Cairo Wastewater Project (GCWWP) presently underway has required deep excavations, frequently penetrating through the confining layer over the aquifer, which necessitates construction dewatering.

Most construction dewatering has been accomplished using deep well systems. Experience has demonstrated that a number of features of the Nile River aquifer system make the interpretation of pumping tests, as well as design and successful implementation of deep well systems, difficult.

Background

The GCWWP represents a multinational public health effort to rehabilitate and upgrade the wastewater system of Cairo. Project design began in 1979 by a consortium of American and British consultants for the Greater Cairo Wastewater Organization, an agency of the Egyptian government. The United States Agency for International Development is funding a significant portion of the project. The project consists of designing and constructing a comprehensive sewerage system that would handle the projected demands of a rapidly increasing population into the 21st century. Currently, the population of Cairo is over 12 million. It is projected to increase to 16 million by the year 2000.

The urbanized portions of the greater Cairo area are divided by the Nile River, which runs in a northerly direction through an otherwise arid region (see Figure 1). The Nile has appropriately been termed the "lifeblood" of Egypt. Population is heavily concentrated on the fertile, cultivated strip along both banks of the Nile. Independent wastewater systems are being constructed on both sides of the Nile to convey wastewater away from the greater Cairo urban areas and toward the deserts that border the fertile alluvial flood plain.

FIGURE 1. Urbanized portion of Cairo along the Nile River.

The GCWWP has required deep, open excavations into alluvial flood plain soils, in some cases up to 12 meters, for the construction of treatment plant facilities, pumping stations, force mains, sewers and collectors. These excavations have extended into the surficial silty clay confining stratum, sometimes requiring penetration into the underlying major sand aquifer.

Since the aquifer is recharged mainly by the Nile, extensive construction dewatering has been necessary. This dewatering is being accomplished most frequently using deep well systems.

During site investigation studies, and as a part of the construction activities, numerous groundwater pumping tests have been conducted on partially penetrating deep wells in order to characterize the hydrogeologic properties of the aquifer and to aid in estimating construction dewatering requirements.

General Aquifer Conditions

Cairo is located on thick alluvial deposits, reported to be up to 500 meters or more in thickness. The alluvial deposits are bounded on the east by limestone cliffs and on the west by the Pyramids Plateau (see Figure 2). The Pyramids Plateau is composed of various sedimentary formations including sandstones, limestones and mudstones, as well as sands and gravels (dune deposits).[1]

A typical soil profile of the project area is shown in Figure 3. The silty clay confining stratum is typically six to ten meters thick, becoming thinner as it moves away from the Nile. This silty clay deposit is fissured and contains occasional layers and seams of sand and silt.

A transition layer, made up of interbedded sand, silt, and clay units and lenses, is generally present between the silty clay stratum and the underlying sand aquifer. The thickness of this

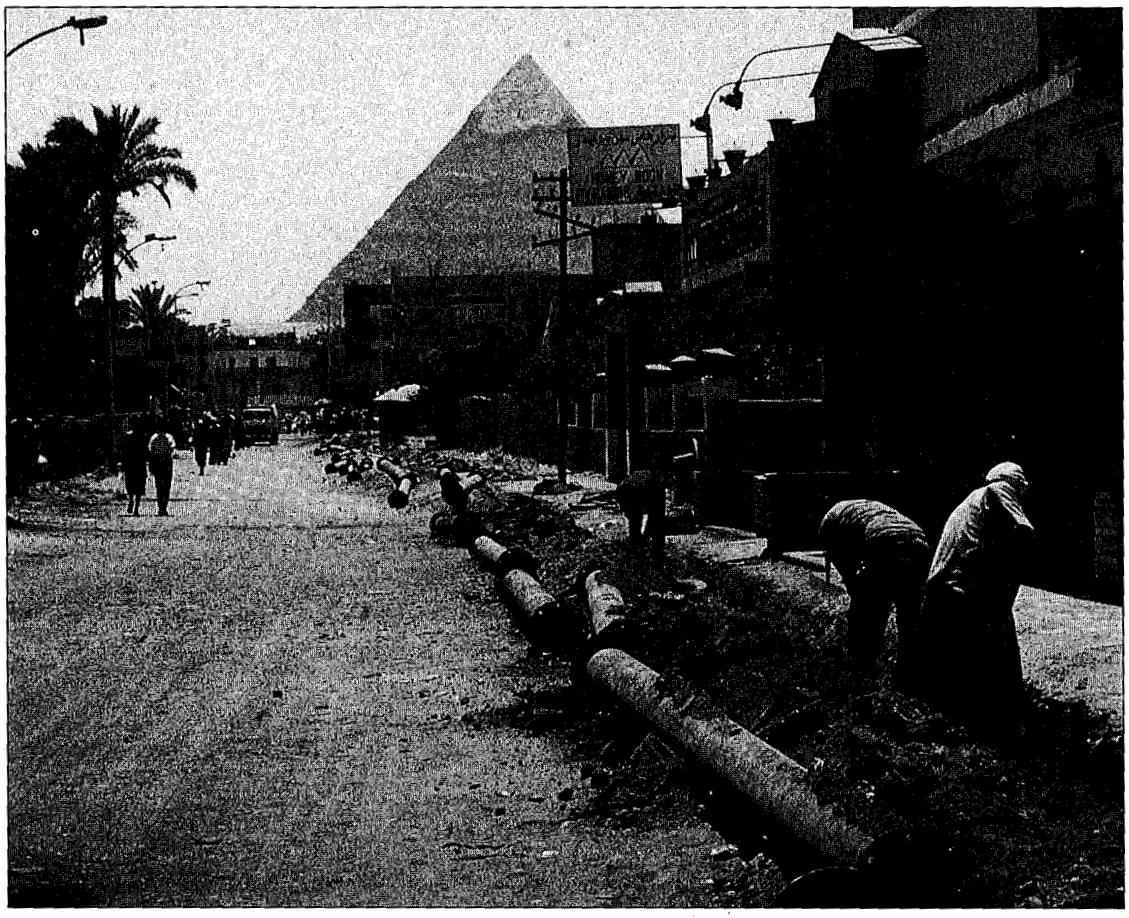

FIGURE 2. The West Bank area of greater Cairo is located in close proximity to the Pyramids Plateau.

transition layer typically varies from zero to about five meters.

The aquifer comprises a medium to fine sand and generally grades coarser with depth to a coarse to medium sand. Gravel, cobbles and discontinuous layers and lenses of silt and clay occasionally occur throughout the aquifer. The aquifer thickness reportedly ranges from 30 to 90 meters in the greater Cairo area and up to 70 meters in the West Bank project area.[1] In a study of Cairo's subsurface geology, Said states that an unconformable continuous layer of plastic clay underlies the sand aquifer.[1] This impermeable lower boundary has been encountered in only a few borings in the West Bank project area, mostly drilled by others before the initiation of the GCWWP. Its presence has also been confirmed in one deep well installation in the village of Nahya for the project, where it was encountered at a depth of approximately 50 meters.

The Cairo aquifer has been described as

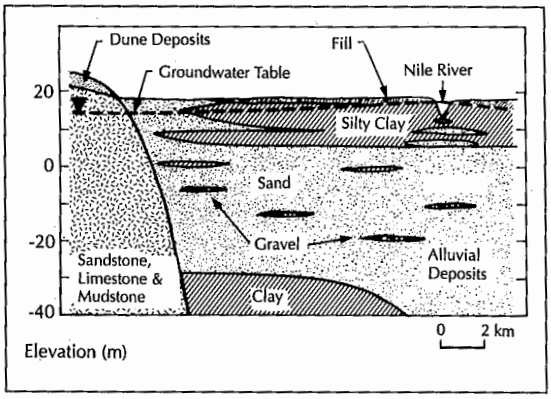

FIGURE 3. Typical soil profile for the West Bank of the Nile River.

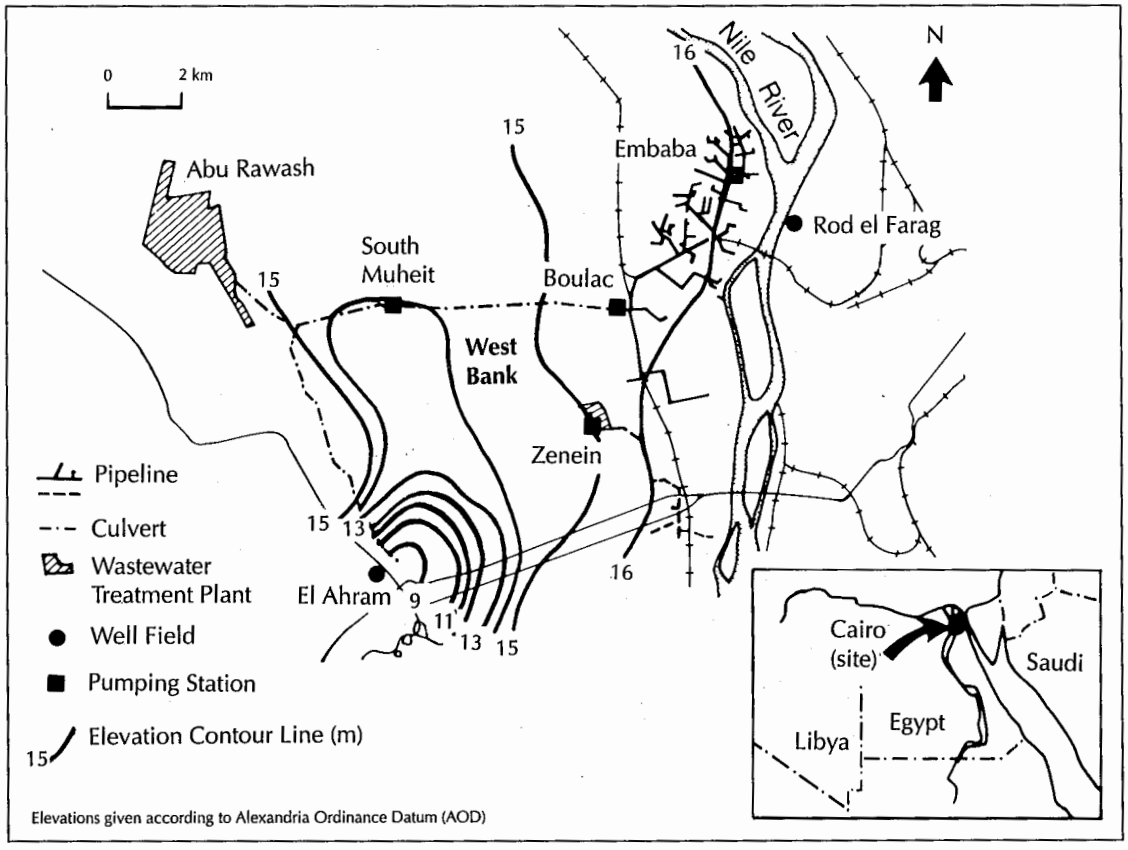

FIGURE 4. Piezometric level contour plan.

"semi-confined" or "leaky."[2] The aquifer piezometric levels are within the confining silty clay stratum, typically to within 2.5 to 3.5 meters of the ground surface. Cracks and fissures within the silty clay, as well as interbedded granular soils, allow some interconnection within the stratum.

A piezometric level contour plan of the area in 1984 (before GCWWP construction) is shown in Figure 4. The piezometric surface is influenced mainly by the Nile and by water supply and irrigation extraction. The piezometric surface slopes away from the Nile, which recharges the aquifer. Nile levels are constant throughout the year except for the month of January, when levels are dropped to permit dredging of the irrigation canal system. Piezometric levels drop during this period.

Also, pumping from various well systems affects the piezometric levels. One major influence on West Bank piezometric levels is the El Ahram well field that pumps continuously, depressing the water table at least seven meters with a radius of influence in excess of four kilometers.

Over 200 soil borings have been drilled in the West Bank project area during the GCWWP. The majority of the borings have been less than 25 meters in depth, and have penetrated only the upper portion of the aquifer. Grain size (D_{10}) analyses performed on soil samples obtained from these borings indicate that the aquifer sand grades coarser with depth (see Figure 5) and the silt content is greater near the top of the aquifer. Deeper borings indicate that gravelly sand is present in isolated lenses, typically in the lower portion of the aquifer, and therefore, the hydraulic conductivity of the aquifer generally increases with depth.

Because of the horizontal bedded nature of the alluvial deposits, the horizontal hydraulic conductivity, K_h, of the aquifer is greater than the vertical hydraulic conductivity K_v. Rahman reported that the average value of isotropy,

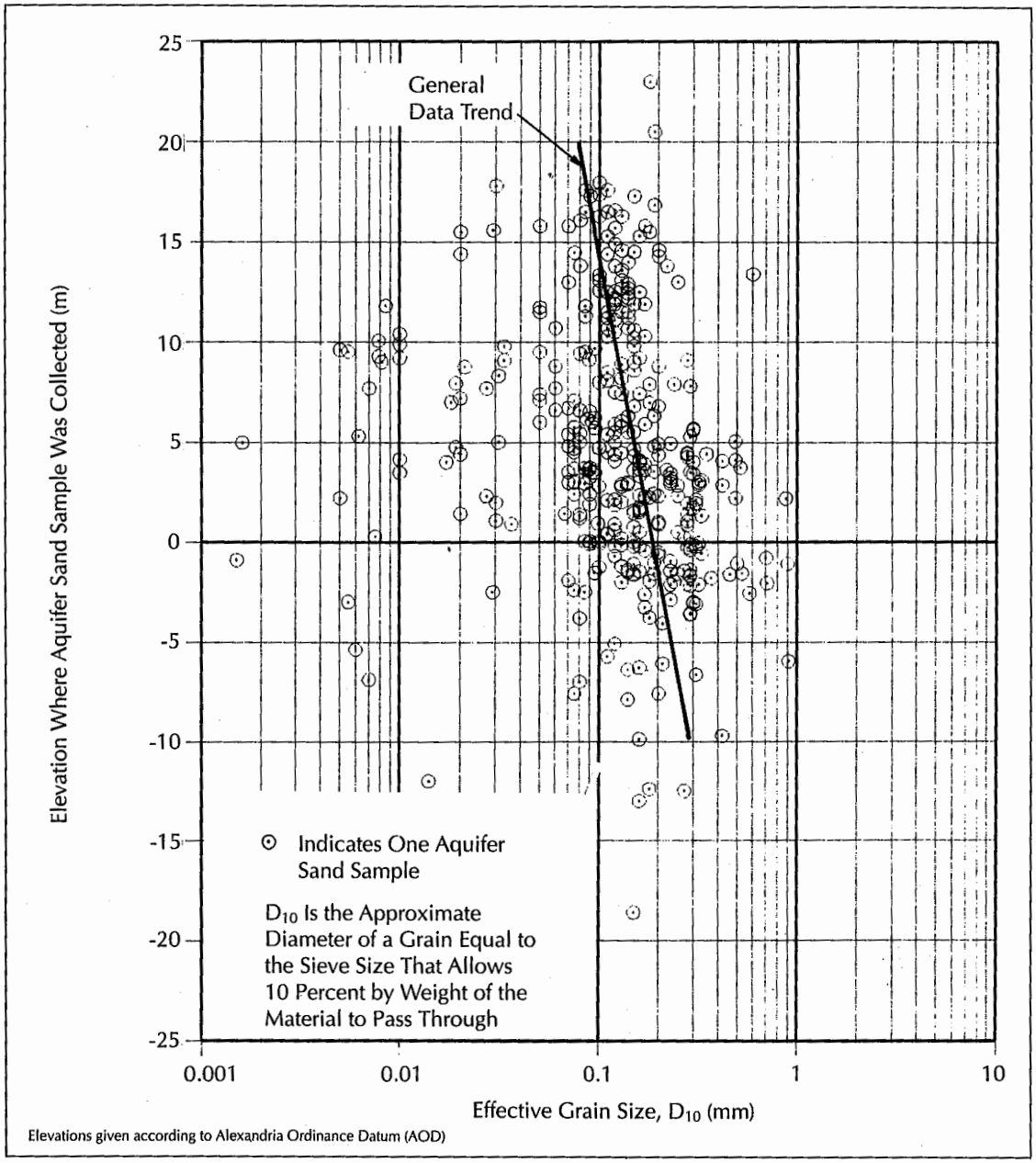

FIGURE 5. Effective grain size, D_{10}, versus elevation.

K_v/K_h, is between 0.1 and 1 for the aquifer in the West Bank area.[3] It is probably considerably larger where interbedding is more prevalent, based on the range of grain sizes that were observed there.

Deep Well Design Considerations

Most excavations required for the GCWWP are within the confining silty clay stratum. Excavation depths up to 12 meters have been required and have extended to nine meters below the preconstruction groundwater table, sometimes penetrating to the top of the sand aquifer. Generally, it has been necessary to lower and maintain groundwater levels at least one meter below the proposed invert prior to excavation, because of the leaky nature of the fissured silty clay confining stratum.

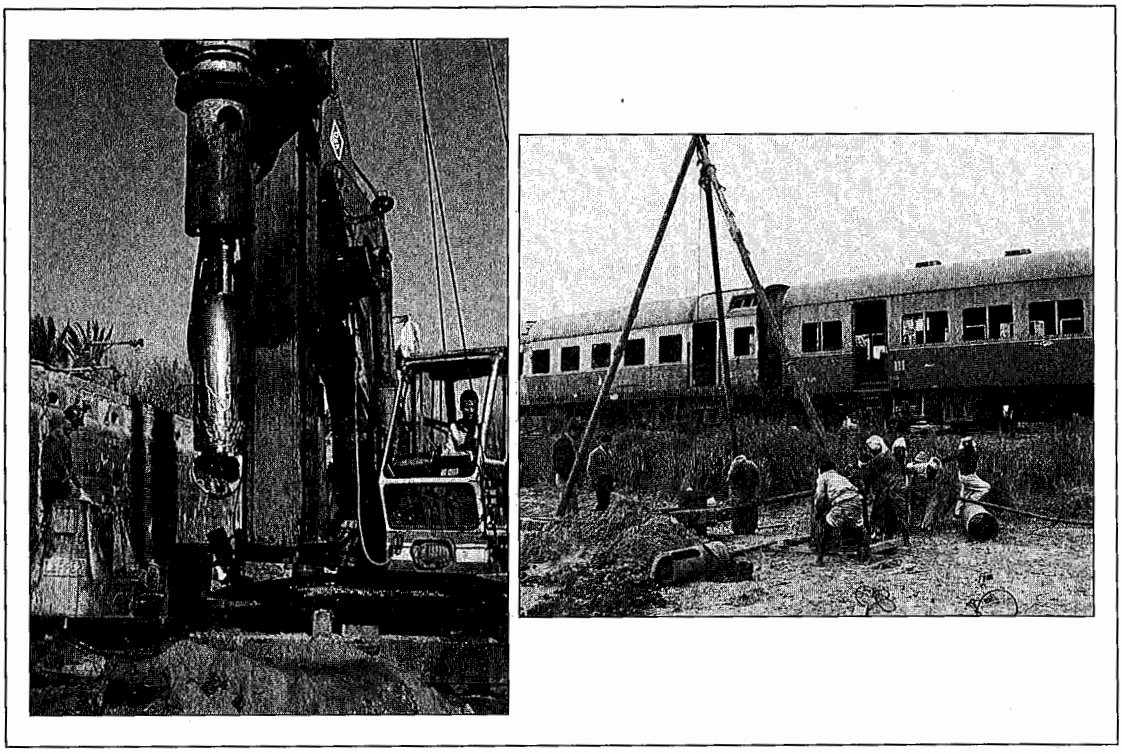

FIGURE 6. Typical well drilling methods used for construction dewatering deep well installation.

Typically, the most effective method of dewatering the Nile aquifer soil has been with deep wells. Fully penetrating wells are generally not economical because they would need to be 50 to 70 meters or more deep, which is beyond the capabilities of most of the locally available drilling equipment (see Figure 6). Partially penetrating deep wells installed with submersible pumps, therefore, have been the main system employed for dewatering the deeper excavations for the project.

Designing a partially penetrating deep well requires experience and substantial information on the hydrogeologic properties of an aquifer. Pumping test data are essential to design adequate dewatering systems. Pumping tests for construction dewatering design should resemble the probable final systems as much as possible in order to provide representative data.

Screen lengths and depths are critical factors in partially penetrating deep well design, especially in the Nile aquifer system where permeability increases with depth. The screens must be installed deep enough to provide sufficient submerged screen length to accommodate the pumping rates required in order to achieve the necessary drawdowns. However, there is a trade-off since installation costs increase substantially with deeper wells, which require more expensive drilling methods and greater material cost. Also, deeper wells produce larger volumes of water because the screens penetrate the higher permeability soils deeper in the aquifer.

Several pumping tests were designed and conducted for the GCWWP using partially penetrating deep wells that were believed to be representative of what would be required for actual construction dewatering for excavations. Results of one pumping test for a partially penetrating deep well are discussed below.

Groundwater Pumping Test Results

One partially penetrating deep well and seven piezometers were installed in 1984 at the Embaba site, as shown in Figure 7. The well, in-

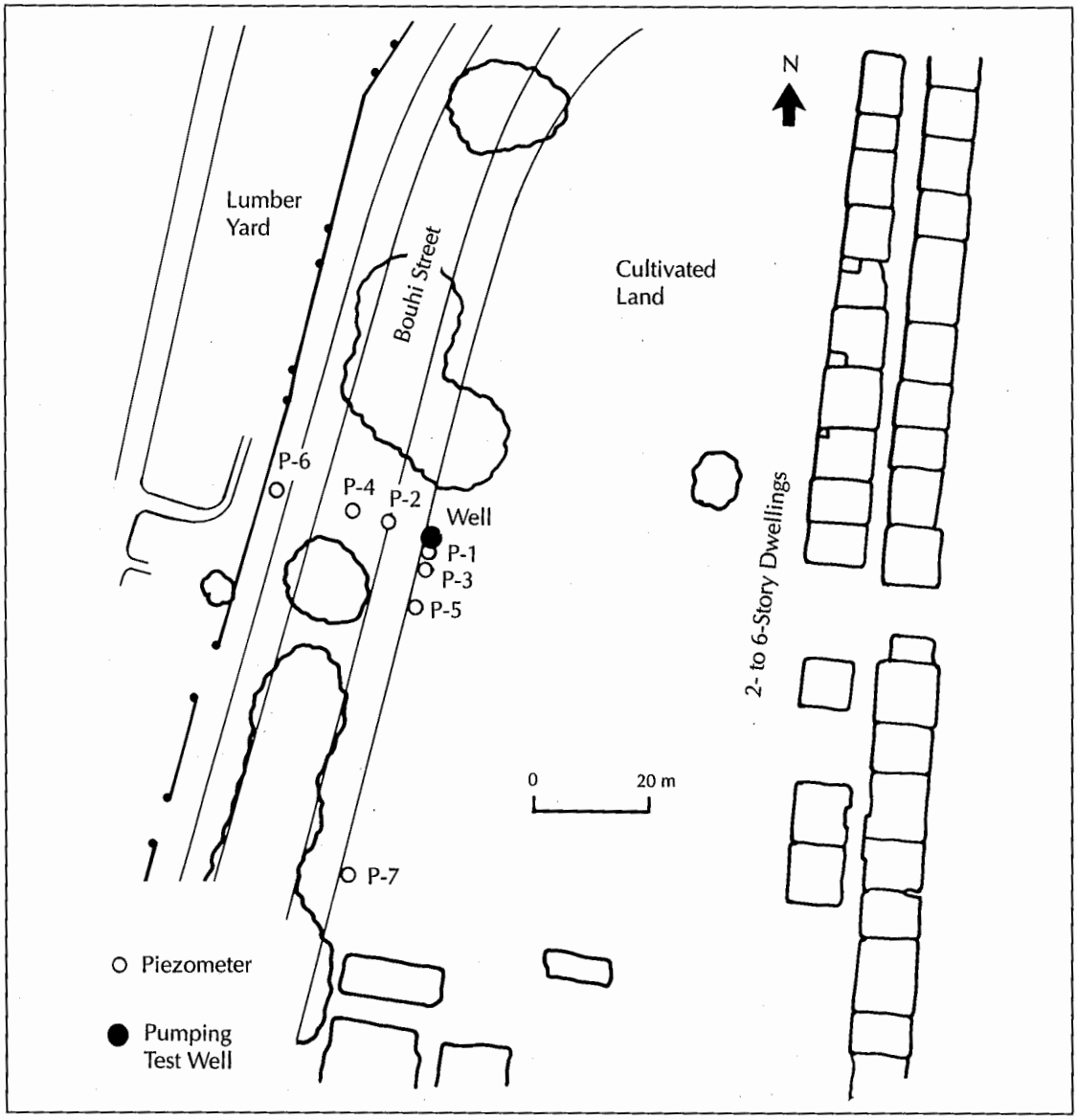

FIGURE 7. Embaba site pumping test set-up.

stalled to a depth of 24 meters, partially penetrated the aquifer, which was estimated to be about 60 meters thick at the site.[1] Installation was accomplished with a rotary drill using a flight auger in clay and a bucket auger in sands while maintaining a positive head of water in the hole. Holes were cased to a depth of about three meters.

The well consisted of a screen 12 meters long and 0.45 meters in diameter, and of a 12-meter blank riser section above the screen.

Six piezometers were installed to depths of about 14.5 meters. Another piezometer was installed to a depth of 17.9 meters. Although not shown in Figure 7, one other piezometer was installed within the 72-millimeter thick annular filter pack of the well, with a tip level just above the top of the well screen. The filter pack of the well was typically a quartz sand, uniformly graded from 0.5 to two millimeters.

A step-drawdown test was conducted at an initial pumping rate of 49 cubic meters per hour (m^3/h) for one hour, followed by recovery for one hour. The sequence was repeated at two

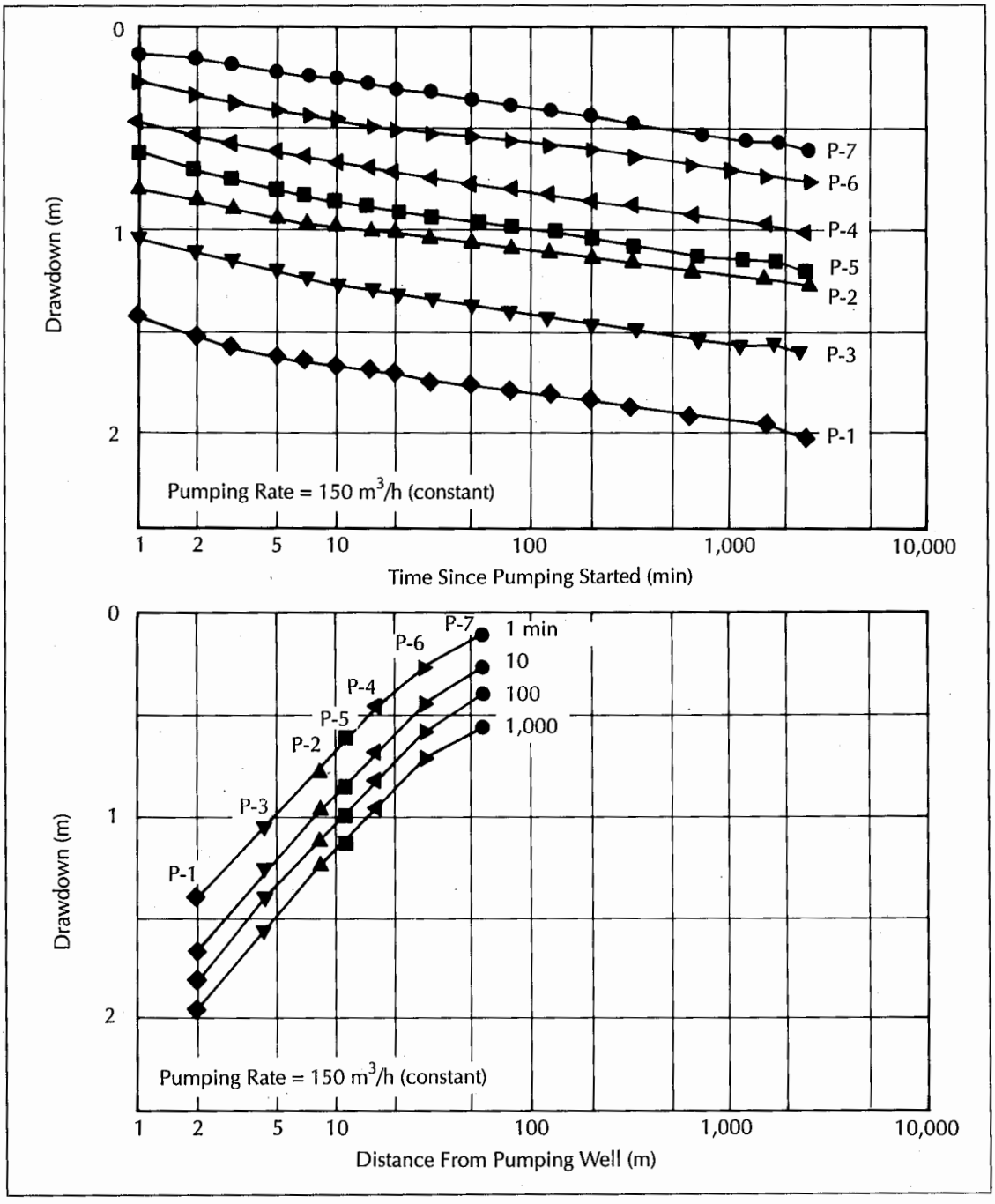

FIGURE 8. Pumping test results from the Embaba site.

additional pumping rates of 113 m^3/h and 145 m^3/h. After completing the step-drawdown test, a long-term test was conducted — the well was pumped at 150 m^3/h for 33 hours, followed by a period of recovery.

During the long-term test, the drawdown in the well ranged from 5.5 to 6.5 meters. The drawdown in the piezometer within the well filter was four to five meters. Drawdown in the piezometers at the end of the pumping period ranged from a minimum of 0.6 meters to a maximum of two meters.

The results of the Embaba pumping test are typical of pumping tests conducted in other areas of the West Bank within the Nile aquifer system. Graphical results of the long-term pumping test are shown in Figure 8.

Simplified Jacob analysis for transient conditions was used to estimate aquifer transmissivity. Such an approach is recommended by Powers[4] and was found to be a reasonable and practical method that was suited for the purpose of assessing construction dewatering requirements. Based on the slope of the drawdown versus log distance plot, a transmissivity on the order of 50 square meters per hour (m^2/h) is suggested. This value is not representative of the aquifer because, for a partially penetrating well, the plot is distorted close to the well due to upward flow. Comparison with actual dewatering system results show this value to be low. The distortion is typically present within a distance equal to about one aquifer thickness away from the well. Since the farthest piezometer was only 55 meters from the pumping well, the curve lies within the area where partial penetration affects drawdown. To interpret such a plot, a correction for partial penetration must be made, or drawdown data must be obtained further from the well. Procedures for correcting the distorted curve based on estimates of isotropy, K_v/K_h, are available as proposed by Jacob[5] and Butler,[6] among others. Results of the drawdown versus log time plot show that the actual transmissivity of the aquifer is about 200 m^2/h. This value agrees well with actual dewatering system results.

Deep Well System Details

From July 1986 through about August 1987, deep well dewatering systems were installed around excavations and operated continuously to permit the construction of four pumping stations on the West Bank portion of the GCWWP.

The general layout for the deep well system used at the Embaba site is shown in Figure 9. Stratigraphic conditions at the site include a silty clay to silt confining stratum that is approximately seven meters thick. Below seven meters, a fine to medium sand aquifer unit was encountered. The aquifer is estimated to be 60 meters thick at the site. The original static groundwater level is about two meters below the ground surface.

The excavation for the pumping station structure was an open cut with sides sloping to almost ten meters below existing groundwater. Fifteen deep wells, each about 27 meters deep, were installed for dewatering this excavation. Eight of these wells were operating during initial dewatering. Piezometers were installed within and around the excavations to monitor the performance of the deep well dewatering system. To estimate flow rate, discharge from the well was routed through a V-notch weir box before emptying into an irrigation drain.

Details on the dewatering systems installed at three other sites in the GCWWP are summarized on Table 1, along with data from the Embaba site. The locations of the sites are shown on Figure 4. Generally, the pumping station excavation geometry was as shown in Figure 9 at all sites except for Zenein, where sheet piles were used around the deepest portion of excavation. A typical dewatered pumping station excavation is shown in Figure 10.

Deep Well System Performance

During 1986 and 1987, the performance of the deep well dewatering systems for the excavations at the four sewage pumping stations (as shown in Figure 4) was monitored. All sites were successfully dewatered, although multiple attempts were required at several sites as a result of the inadequacy of the initially installed systems.

Problems resulted from initial underestimation of flow quantities, not installing well screens deep enough and/or lack of proper well development. Estimated peak flows for the final successful dewatering systems and actual aquifer parameters based on dewatering results are shown in Table 2.

Of particular interest in comparing pumping test results to dewatering system results are the drawdown versus log distance curves obtained for each deep well system. A plot of drawdown versus distance from the centroid of pumping is shown in Figure 11 for each of the four sites. A bend (distortion) in all of the curves is noted at a distance of approximately 50 to 100 meters from the centroid of pumping, corre-

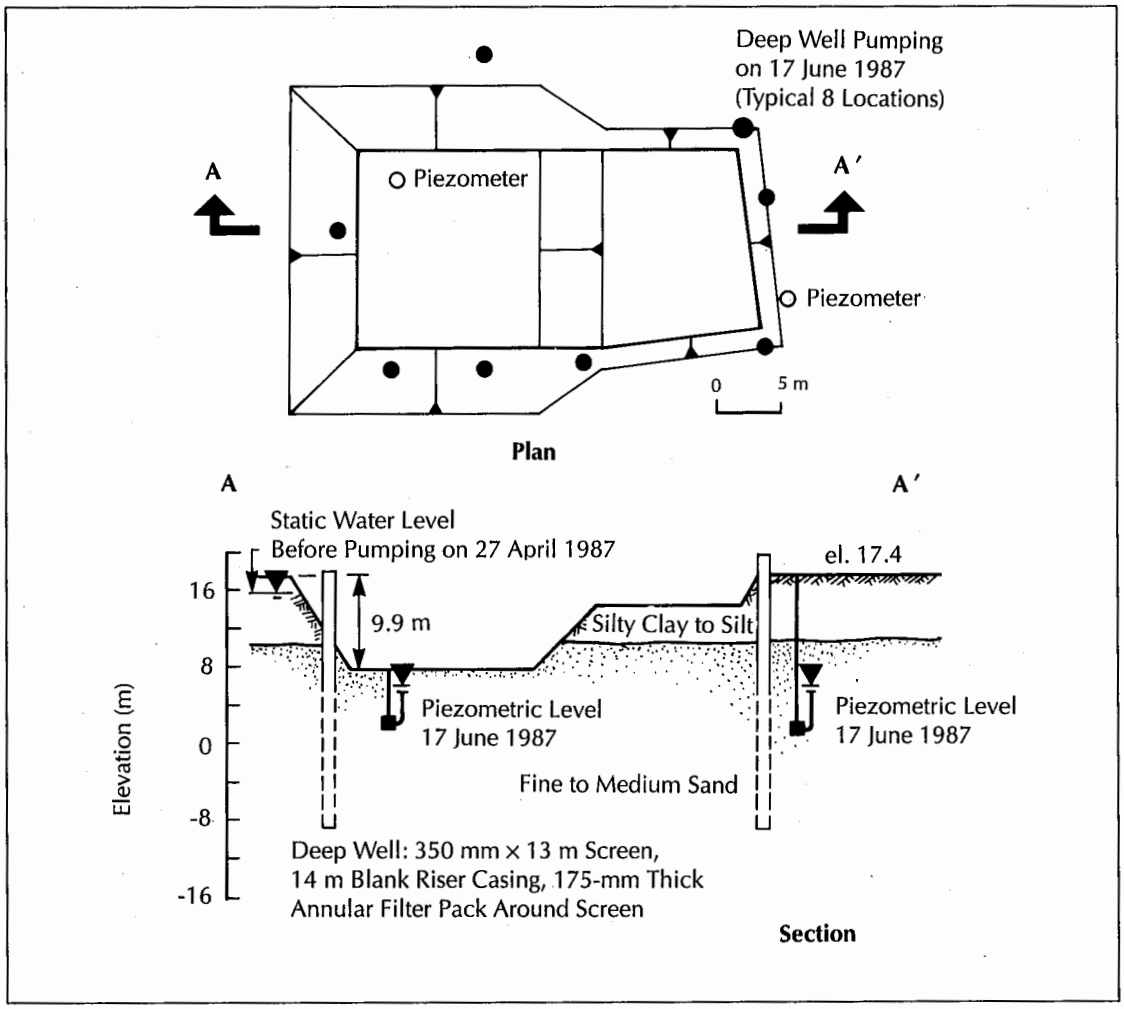

FIGURE 9. The deep well system layout for the Embaba site.

TABLE 1
Deep Well Dewatering System Details

	Boulac	Embaba	South Muheit	Zenein
Ground Surface Elevation, m	17.5	17.4	16.4	18.0
Subgrade Elevation, m	5.7	7.5	4.9	6.3
Excavation Depth, m	11.8	9.9	11.7	11.7
Elevation Top of Aquifer, m	5.7	10.0	6.0	5.0
Estimated Aquifer Thickness, m	40	60	50	70
Initial Groundwater Elevation, m	14.5	15.5	13.0	14.5
Number of Deep Wells Installed	23	15	31	10
Depth of Wells, m	27	27	19-29	25-29
Deep Well Screen Length, m	15	13	7-13	8-11
Deep Well Boring Diameter, m	1.0	0.7	0.7-1.0	0.4
Deep Well Casing Diameter, m	0.35	0.35	0.35	0.3

FIGURE 10. Dewatered excavation for a West Bank Pumping Station (the Boulac site).

TABLE 2
Deep Well Dewatering System Results

	Boulac	Embaba	South Muheit	Zenein
Number of Deep Wells Operating at Peak Discharge	13	8	23	10
Estimated Peak Discharge, m^3/h	960	1,150	2,530	1,930*
Estimated Aquifer Thickness, m	40	60	50	70
Approximate Drawdown at Centroid of Pumping System, m	8.9	9.7	10.8	9.3
Estimated Radius of Influence of Dewatering System, m	2,500	1,500	2,500-5,000	3,000
Pumping Duration, days	70	90	130	285
Calculated Aquifer Transmissivity, m^2/h	140	160	390	570
Hydraulic Conductivity, m/sec	1×10^{-3}	7×10^{-4}	2×10^{-3}	2×10^{-3}

* Includes 160 m^3/h from wellpoints.

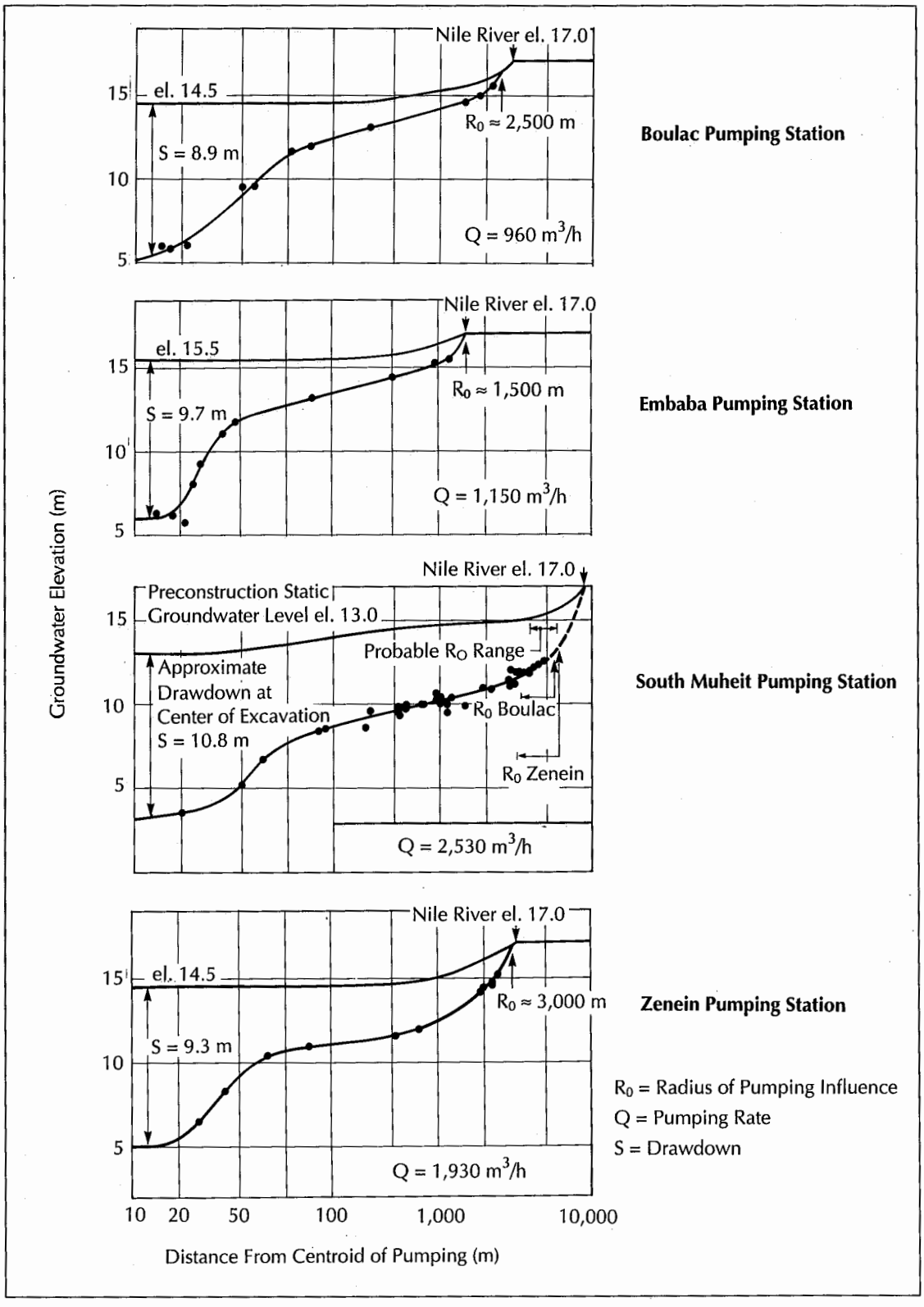

FIGURE 11. Deep well system drawdown versus log distance for each site.

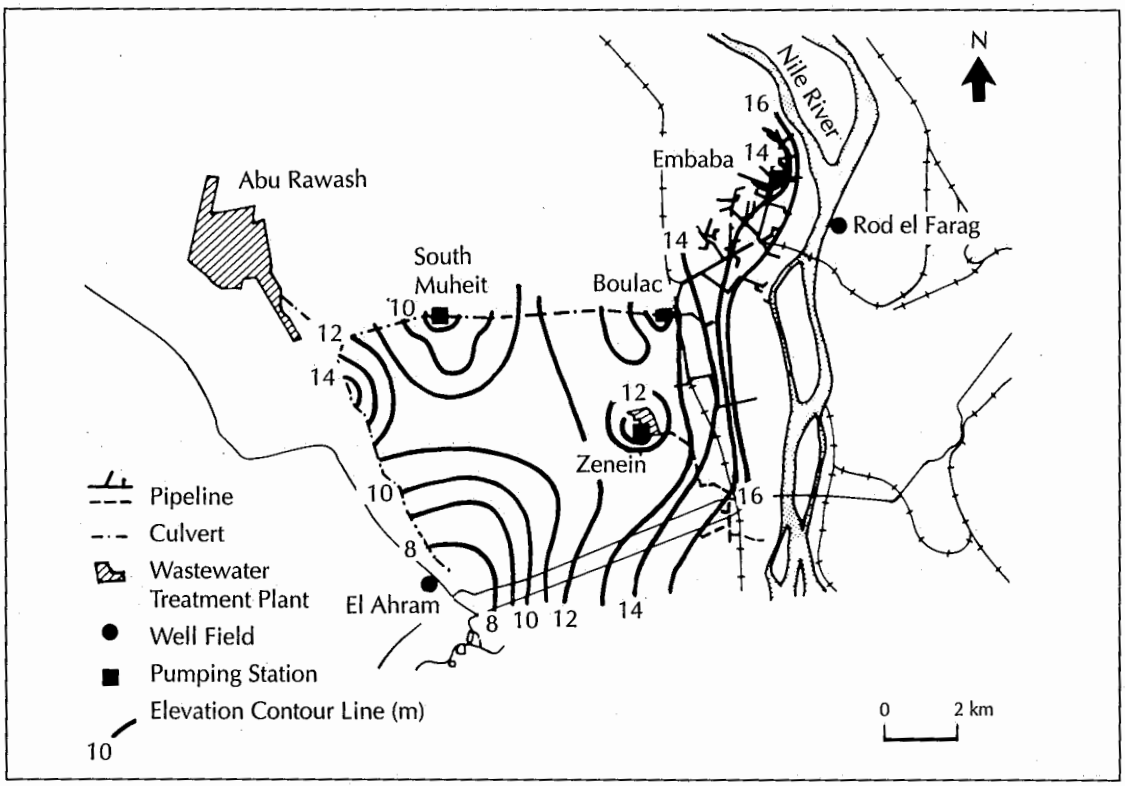

FIGURE 12. Piezometric level contour map during construction dewatering in July 1987.

sponding to a distance of about one to two aquifer thicknesses. Beyond this point, partial penetration effects are negligible since flow is predominantly radial in direction. Simplified Jacob analysis of the drawdown versus log distance data agrees well with the analysis of the drawdown versus log time data.

Another significant feature of the drawdown curves is the large radii of pumping influence, R_o. Estimated R_o values for the sites range between 1,500 meters and approximately 5,000 meters. Figure 12 shows a piezometric level contour map based on measurements obtained during July 1987, when dewatering was in progress at all sites. When compared to the preconstruction piezometric level contour plan, Figure 4, this figure demonstrates the profound influence that dewatering pumping had on piezometric levels in the entire West Bank area. The cones of depression overlapped. Peak discharge rates were estimated to range from 960 to 2,530 m^3/h. These flow rates are comparable to pumping rates for the public well fields at El Ahram, also shown in Figure 12.

Results

Values of transmissivity have been recalculated for dewatering systems at the four sites, based on modeling the group of wells as a single well with an equivalent radius. The slope of the drawdown versus log distance curve at a distance greater than 100 meters has been used to avoid the influence of partial penetration. Calculated transmissivity values range from about 140 to 570 m^2/h. The value calculated for the Embaba site, 160 m^2/h, closely agrees with the value calculated from the time-drawdown pumping test data at the same location.

More sophisticated groundwater models are available for calculating transmissivity in partially penetrating confined aquifers. Methods have been proposed by Jacob,[7] Sternberg as reported by Bouwer,[8] and Streltsova.[9] These methods require a reasonably close estimate of aquifer thickness and the degree of anisotropy of the aquifer. Application of these models has been performed by Rahman[3] and is beyond the scope of this study.

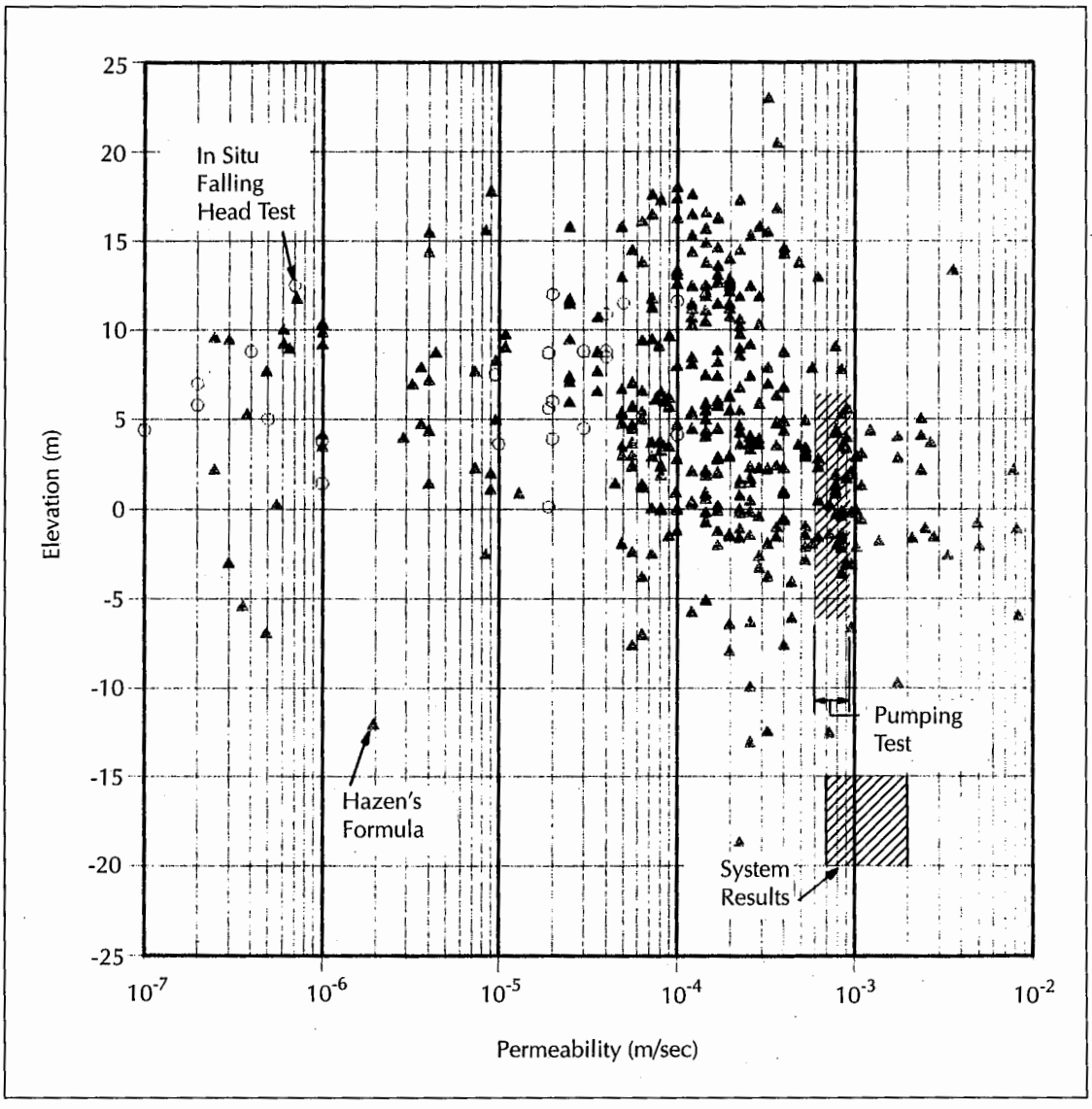

FIGURE 13. Summary of permeability data.

The point at which the drawdown-distance curve bends (distorts) because of partial penetration can also be used in order to estimate the anisotropy in permeability of the aquifer. The study by Rahman indicates values of K_v/K_h on the order of 0.1 to 1.0 for the aquifer.[3] Based on the vast amount of grain size data from the GCWWP reviewed, it is believed that, although the upper portions of the aquifer probably have a higher ratio of anisotropy and are less permeable, the aquifer becomes more permeable and less anisotropic with depth.

Figure 13 presents estimated soil permeability ranges based on various methods of determination. A large scatter in data is apparent with estimates from in situ falling head tests significantly underpredicting permeability compared to actual system results by up to several orders of magnitude. Estimates from grain size correlations can also be misleading, especially due to the lack of data from lower in the aquifer where deep well screens are typically located. Unless there is an understanding of the hydrogeology of the Cairo aquifer, the use of such data can lead to large errors in predicting dewatering system requirements,

due to the anisotropy and the increasing permeability with depth. For the Nile aquifer system, a properly performed pump test is the most appropriate method for estimating dewatering requirements.

Conclusions

Design of a partially penetrating deep well dewatering system in the Cairo West Bank area requires an understanding of the unique features of the Nile River aquifer system and the consequences of partial penetration of wells. A properly designed and performed pumping test program is recommended for estimating dewatering requirements.

The pumping test should be representative of the actual dewatering system anticipated at a particular site. It is further recommended that some of the monitoring piezometers be installed at a distance of at least 200 meters (two aquifer thicknesses) away from the pumping well to provide drawdown data at a distance where predominantly radial flow is expected. The data and drawdown data from piezometers can be used to construct the complete distance-drawdown curve, providing a realistic estimate of aquifer transmissivity. If piezometers cannot be installed at this distance, then time versus drawdown data should be used to predict transmissivity.

If the pumping test deep well is representative of an actual system component and is performed at comparable pumping rates, then the pumping test results can be used directly to "calibrate" an analytical model used for design. A complete distance-drawdown curve from a reliable aquifer pumping test can be used to predict the number of deep wells required to dewater a particular site, based on cumulative drawdown. Depending on the experience of the designer, this approach may be more appropriate than attempting to apply one of the available design models for partially penetrating deep well dewatering systems.

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ROBIN DILL *received an S.B. degree in Geotechnical Engineering from the Massachusetts Institute of Technology, then a Master of Engineering degree from the University of California at Berkeley in 1979, and since that time has been employed by Haley & Aldrich, Inc. He relocated to Egypt from January 1986 though March 1989 to serve as Senior Geotechnical Engineer for the American-British Consultants consortium during the West Bank construction phase of the Greater Cairo Wastewater Project. He is a registered professional engineer and is currently a project manager with Haley & Aldrich. His experience includes construction dewatering as well as underground construction and tunneling.*

MARK M. PETERSEN *received his B.S. and M.S. degrees in Civil Geotechnical Engineering from the University of Illinois in 1980 and 1981, respectively. He was a geotechnical engineer on the Greater Cairo Wastewater Project from 1986 to 1989. He is currently a Geotechnical Engineer for Black & Veatch's Energy Group, working on a variety of power projects. He is a registered professional engineer in the state of Kansas.*

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Design of the High Street Ramp, Boston

The challenges of a confined site for a temporary highway exit ramp were met by supporting several of its piers on the substructure of a nearby office tower.

ABDOL R. HAGHAYEGHI &
PETER J. QUIGLEY

The construction of one of the largest office complexes in downtown Boston, International Place, necessitated replacing the existing High Street ramp on the Central Artery. The main components of the complex are two cylindrical towers that were constructed at different times. The first tower, One International Place, was completed in 1988 and is 46 stories high. The second, recently completed tower, Two International Place, is 35 stories high. The location of the second tower required the removal of an existing southbound off-ramp from the elevated Central Artery, and thus the construction of a temporary new ramp to serve this portion of the Artery until the new depressed Central Artery is constructed underground.

In addition to carrying a heavy urban traffic flow, this temporary ramp had to be designed in order to accommodate the future construction of the depressed Central Artery. The design for this ramp was based on a sequence of construction that allowed the uninterrupted operation of the elevated Central Artery and the continuous use of the existing ramp until traffic could be redirected to the new ramp (see Figure 1).

Given the site constraints, five of the eight new ramp piers had to be supported on the substructure of the first tower of International Place. This type of support, and the resulting techniques used during construction, created some unusual challenges for both the designer and contractor.

Highway Design Considerations

The main challenge of this project was to position the new temporary ramp so that it would have minimal impact on the One International Place building and the future depressed Central Artery. In addition, the new ramp had to be designed with a profile and plan layout that would:

- Maintain a minimum pavement width of 22 feet;
- Maintain automobile and truck access to One International Place (see Figure 2);
- Maintain adequate roadway clearances over the city streets below;
- Provide acceptable gradient and stopping sight distance; and,
- Achieve acceptable depth to span ratios

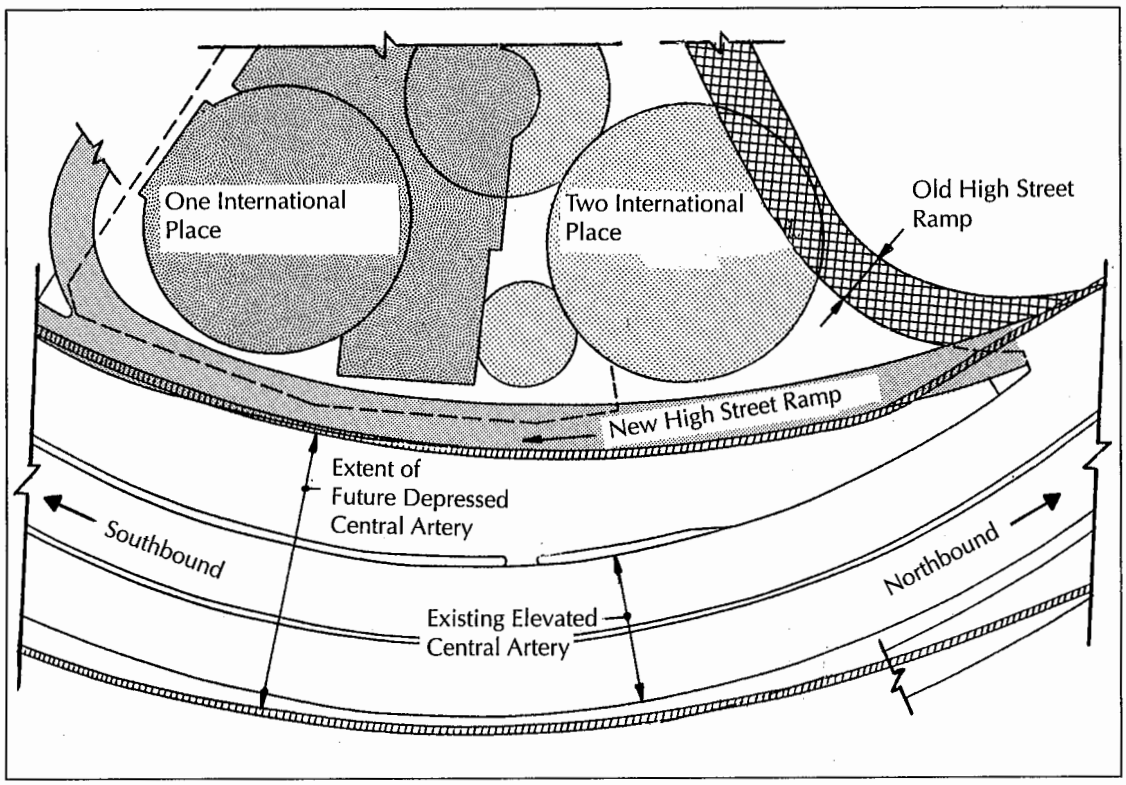


FIGURE 1. Site plan of International Place.

for the ramp girders while meeting all of the other demands.

Once these concerns were addressed and the ramp piers were located, a support scheme had to be developed for each pier that would minimize undesirable vibration and settlement effects on the One International Place building and that would accommodate the construction of the depressed Central Artery. The depressed highway will pass within five to 15 feet of the One International Place building's foundation wall.

After reviewing several different support schemes, it became evident that due to the lack of sufficient space for the new pier foundations, five of the eight ramp piers would have to be located within the basement of the One International Place building. The three remaining piers were supported outside of the building.

Structural Design Considerations

The three ramp piers outside the International Place building are supported on deep foundations. Reinforced concrete appeared to be the most appropriate choice of material for the construction of these outside piers since it is the alternative that requires minimum maintenance during the life of the piers. Each of the three concrete hammerhead piers is supported on six 42-inch diameter caissons that extend not less than 15 feet below the limits of excavation for the depressed Central Artery. These 90-foot long, heavily reinforced caissons were installed using the slurry caisson method. Each concrete pier was designed for the appropriate AASHTO loading requirements under two separate sets of conditions:

- The first was the *current condition* with soil present around the pile cap and the caissons (see Figure 3).
- The second was the *future condition,* when the excavation for the depressed Central Artery will have eliminated the soil that is the only means of lateral support around the caisson cap and caissons (see Figure 4).

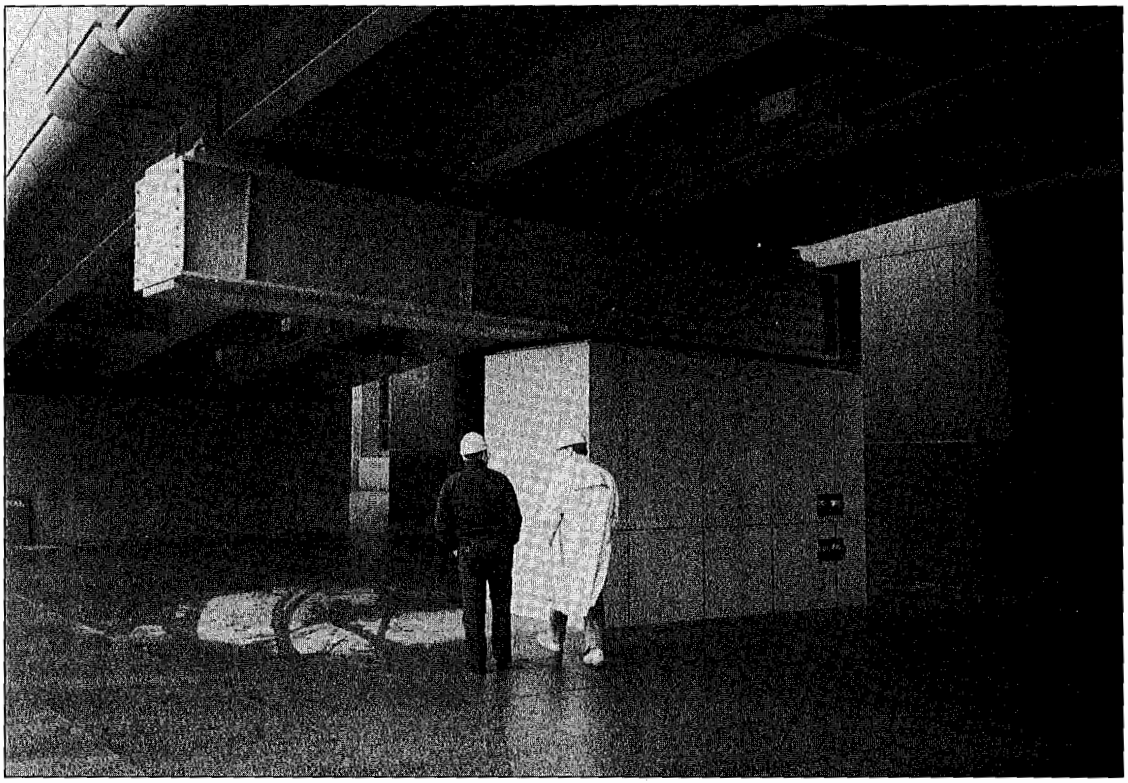


FIGURE 2. View of the ramp at the entrance to the garage at the One International Place building.

Based on the those criteria, each caisson was designed to withstand an axial compressive force of 400 kips, and a bending moment of 800 kip-feet.

For the first condition, a computer model was developed in order to simulate the lateral earth support. This computer model made use of spring supports around the caissons and the pile cap.

For the second condition, a bracing system was schematically designed in order to brace the caissons and the caisson cap. The bracing system will be installed in the future in a step-by-step sequence during the excavation of the depressed Central Artery (see Figure 4).

The remaining five piers were supported inside the One International Place building. In evaluating the possible use of reinforced concrete versus structural steel for these five piers, the following issues were considered:

- Construction schedule and cost;
- Minimum loss of space and disruption within the One International Place building;
- The ability to properly isolate ramp vibrations from the building; and,
- The least impact on the One International Place building during the future demolition of this ramp.

Based on the those considerations, a structural steel system was clearly advantageous. Four of the five steel piers consisted of a single box column supporting a cantilevered box girder (see Figure 5). The fifth pier required two columns. Each of the box columns is supported on a new spread footing bearing on a glacial till subgrade below the lowest level of the existing building. The new spread footings are of a type similar to those used for the existing One International Place building but were designed to function independently from the existing building foundations.

The design forces on the ramp created bending moments in the piers' cantilevered girder

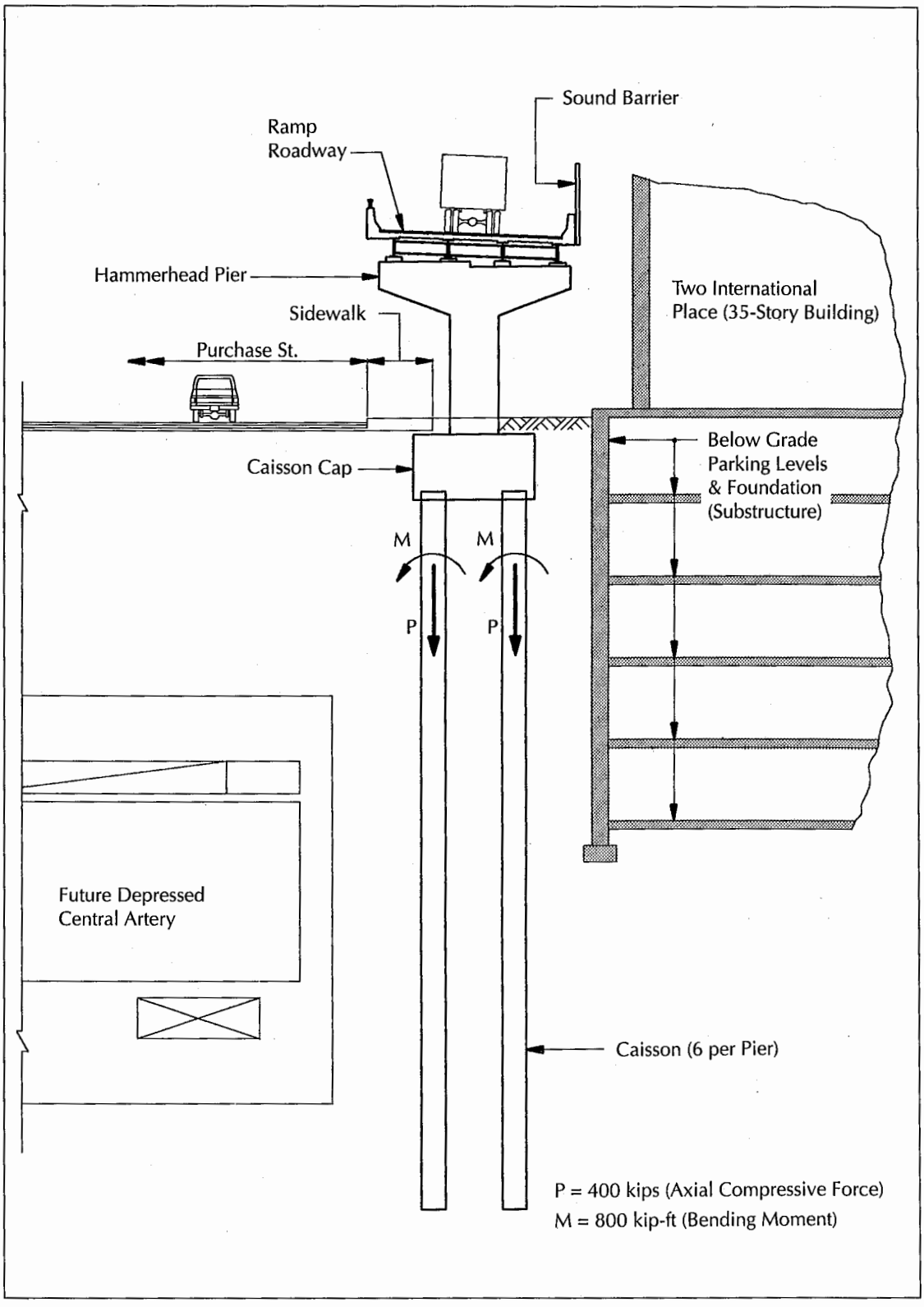


FIGURE 3. Typical cross section of the reinforced concrete pier.

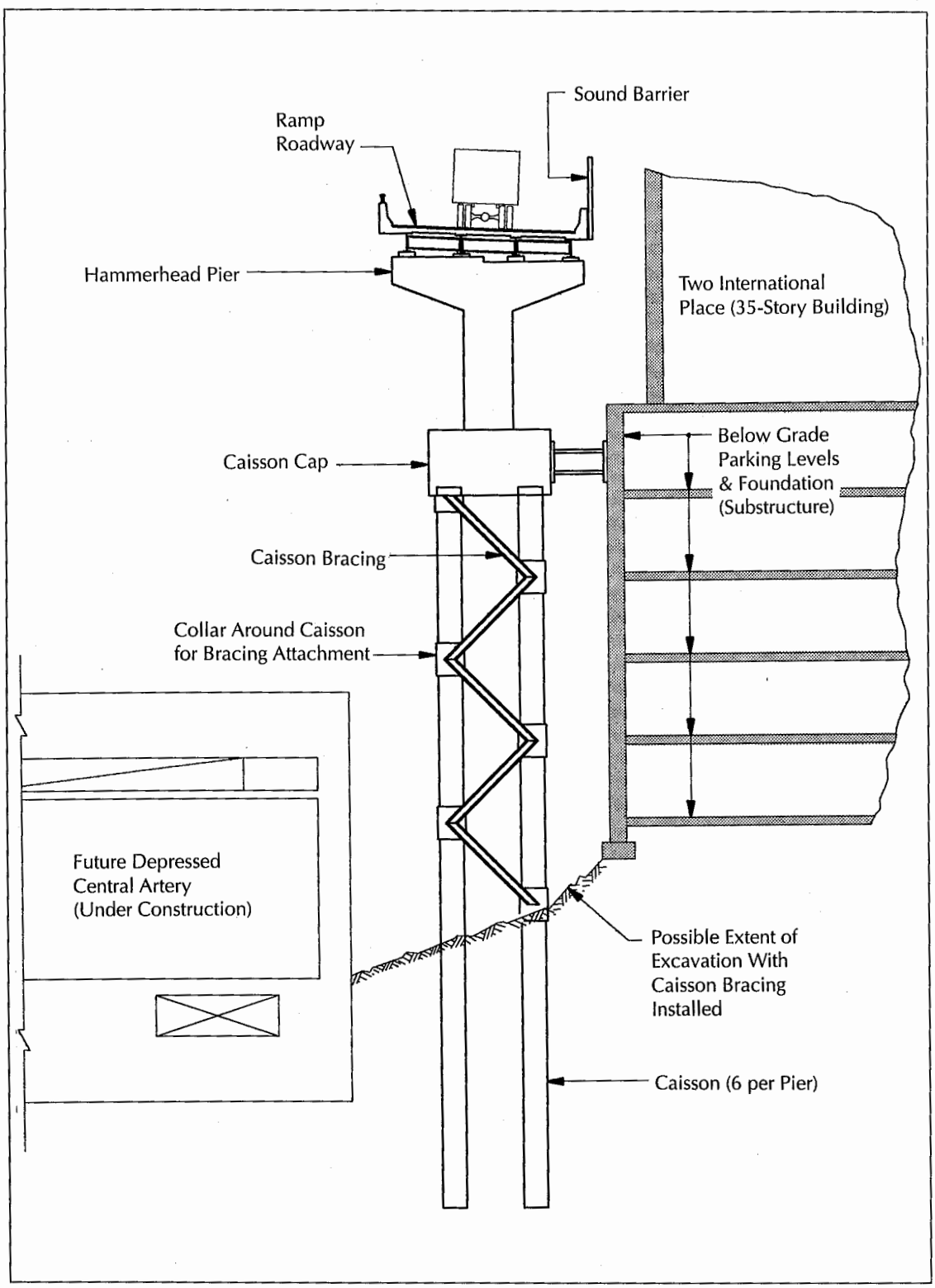


FIGURE 4. Cross section of the reinforced concrete pier during the construction of depressed Central Artery.

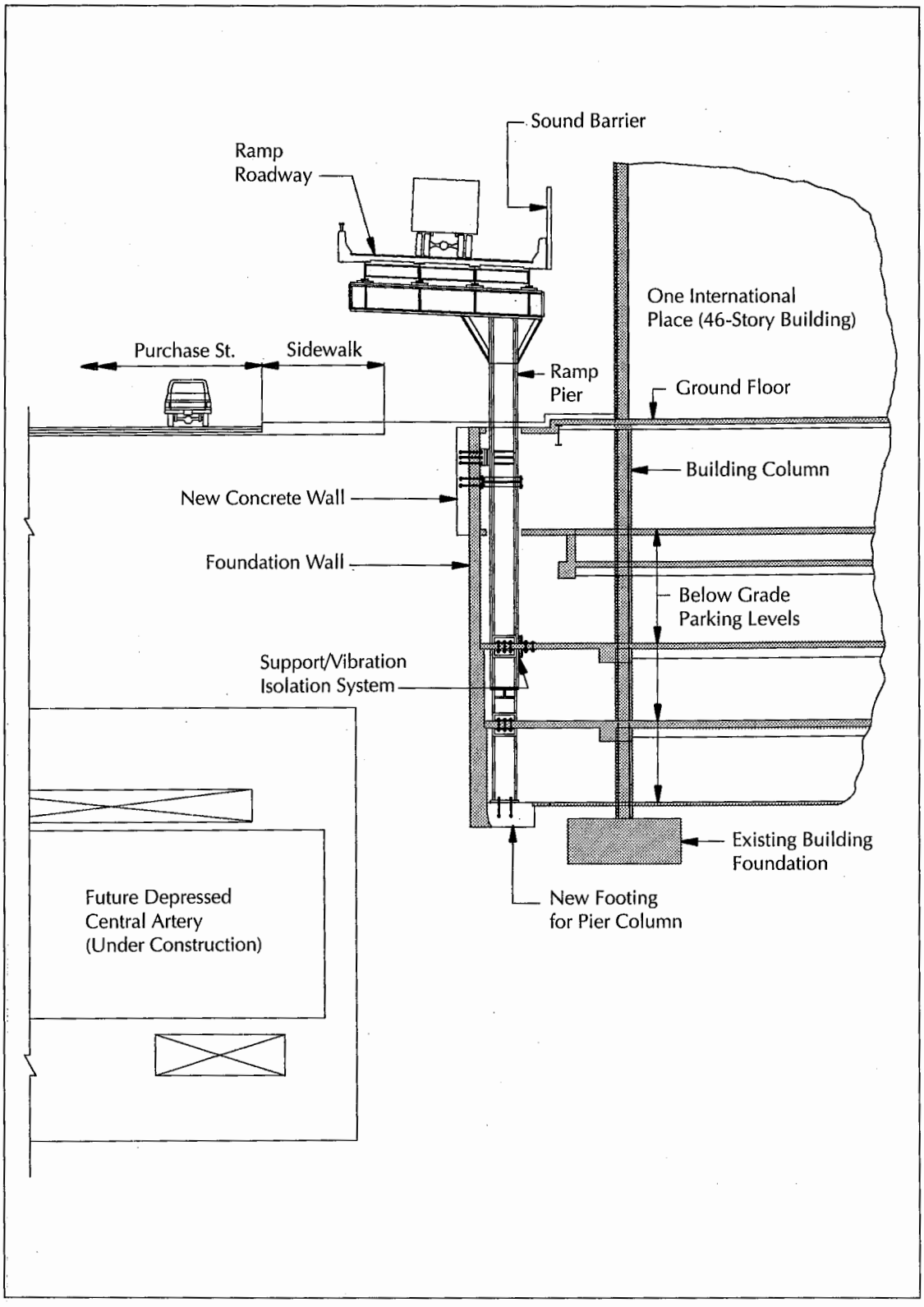


FIGURE 5. Typical cross section of the structural steel pier.

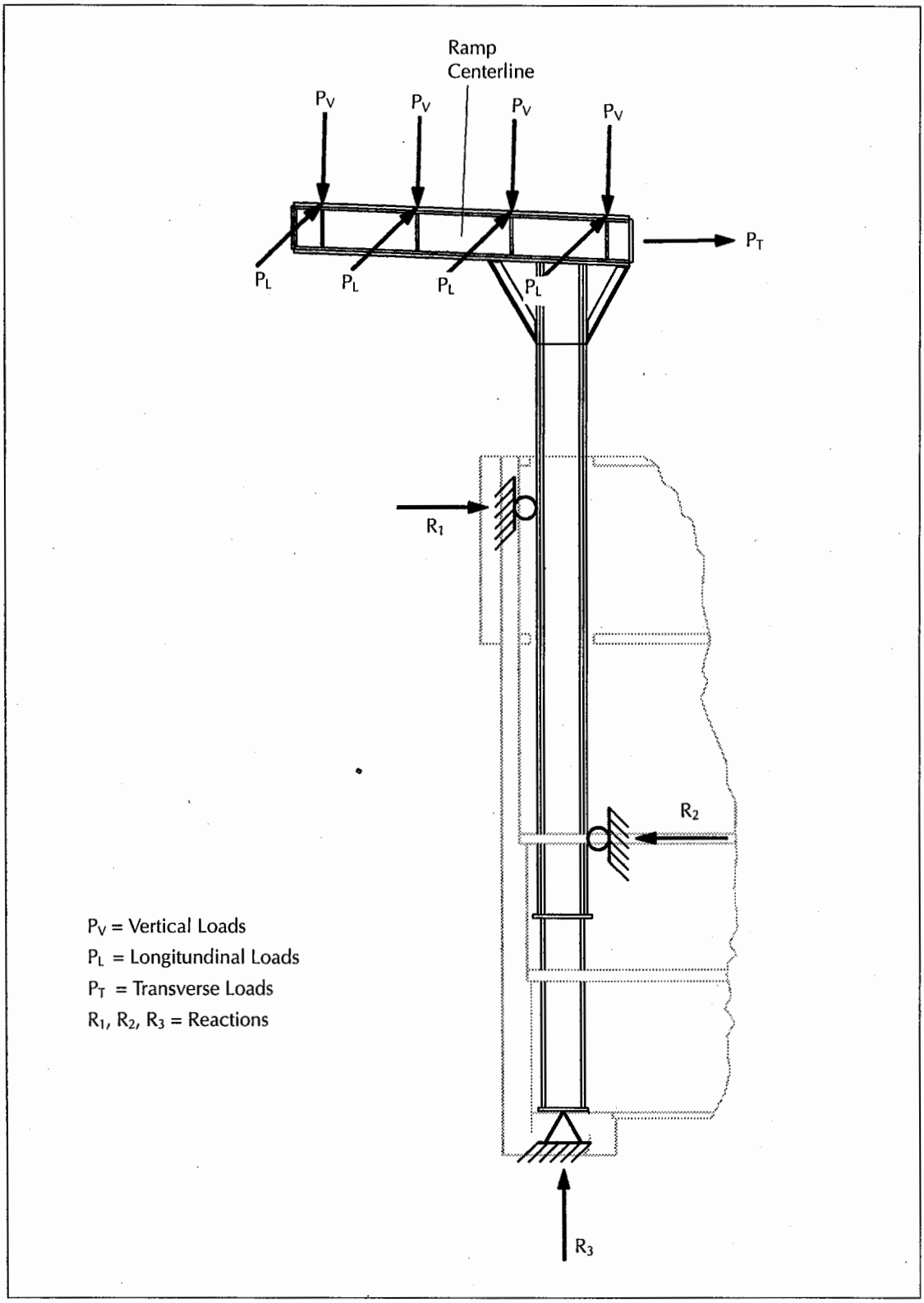


FIGURE 6. Model of the support system for the single column steel piers.

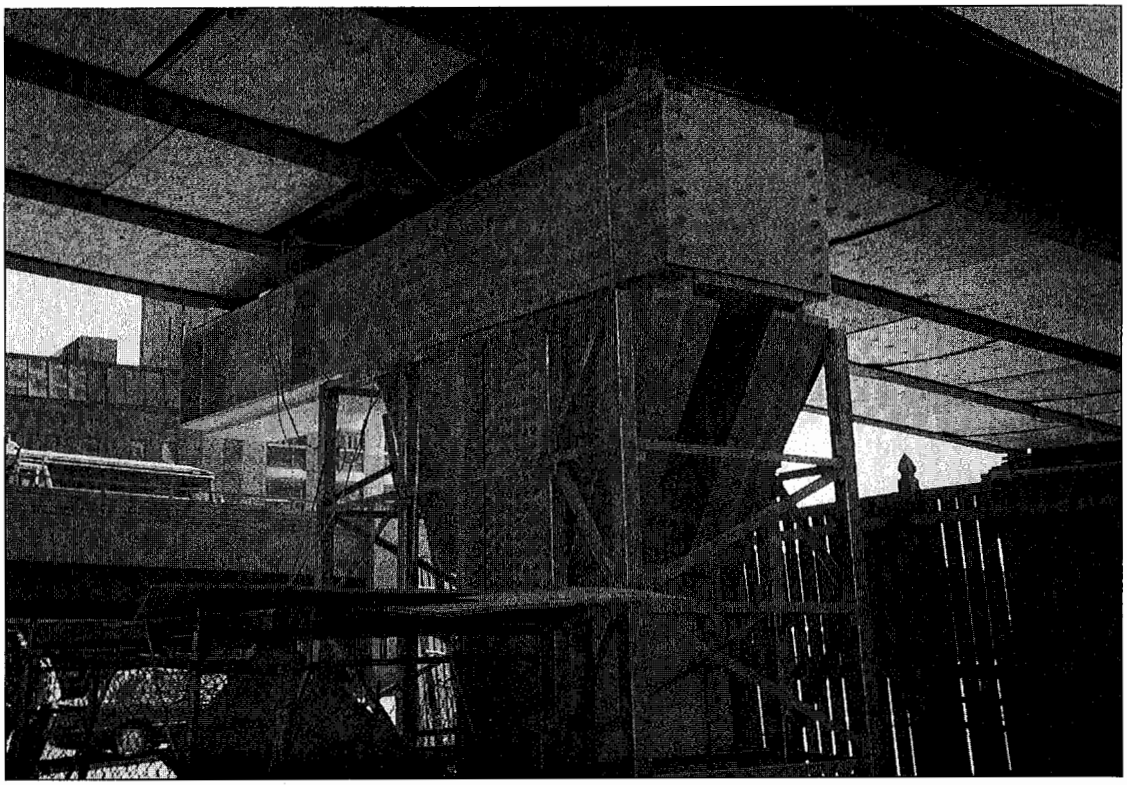


FIGURE 7. A view of a steel pier during construction.

and supporting columns. The moments in the columns were resisted by the garage floors and the foundation wall of the One International Place building acting as a couple (see Figure 6). The design loads creating these bending moments consisted of:

- Vertical loads — dead, live, and impact
- Longitudinal loads — live, seismic, wind
- Transverse loads — centrifugal, seismic, wind

The vertical loads and bending moments on the box column were resisted by the reactions — R_1, R_2 and R_3 — shown in Figure 6. The existing building's foundation wall was reinforced in order to develop reaction R_1, while the reaction R_2 was resisted by the existing garage slabs.

Of particular interest was the transmission of out-of-plane torsional forces from the box column to the garage structure. For these torsional forces, a special collar was designed that connects to the foundation wall and restrains the torsional forces of the box column.

All cantilevered box girders and box columns consisted of two W36 beams with 2.5-inch thick top and bottom cover plates. Many internal stiffeners were used in the box girders in order to increase the rigidity of the section against torsional warping. No field welding was permitted by the Massachusetts Highway Department (MHD) Standard Specifications since fracture-critical steel was used for all five of the steel piers. All field connections were therefore bolted connections that had been preassembled in the shop in order to ensure the proper fitting of all elements in the field (see Figure 7).

The Vibration Isolation System

The most important aspect of connecting the ramp structure to the building structure, however, was the design of the vibration isolation system. During the initial phases of this project, it became apparent that the ramp traffic creates an impact load on the piers that may be transmitted to the office tower through the building substructure. This transfer was completely un-

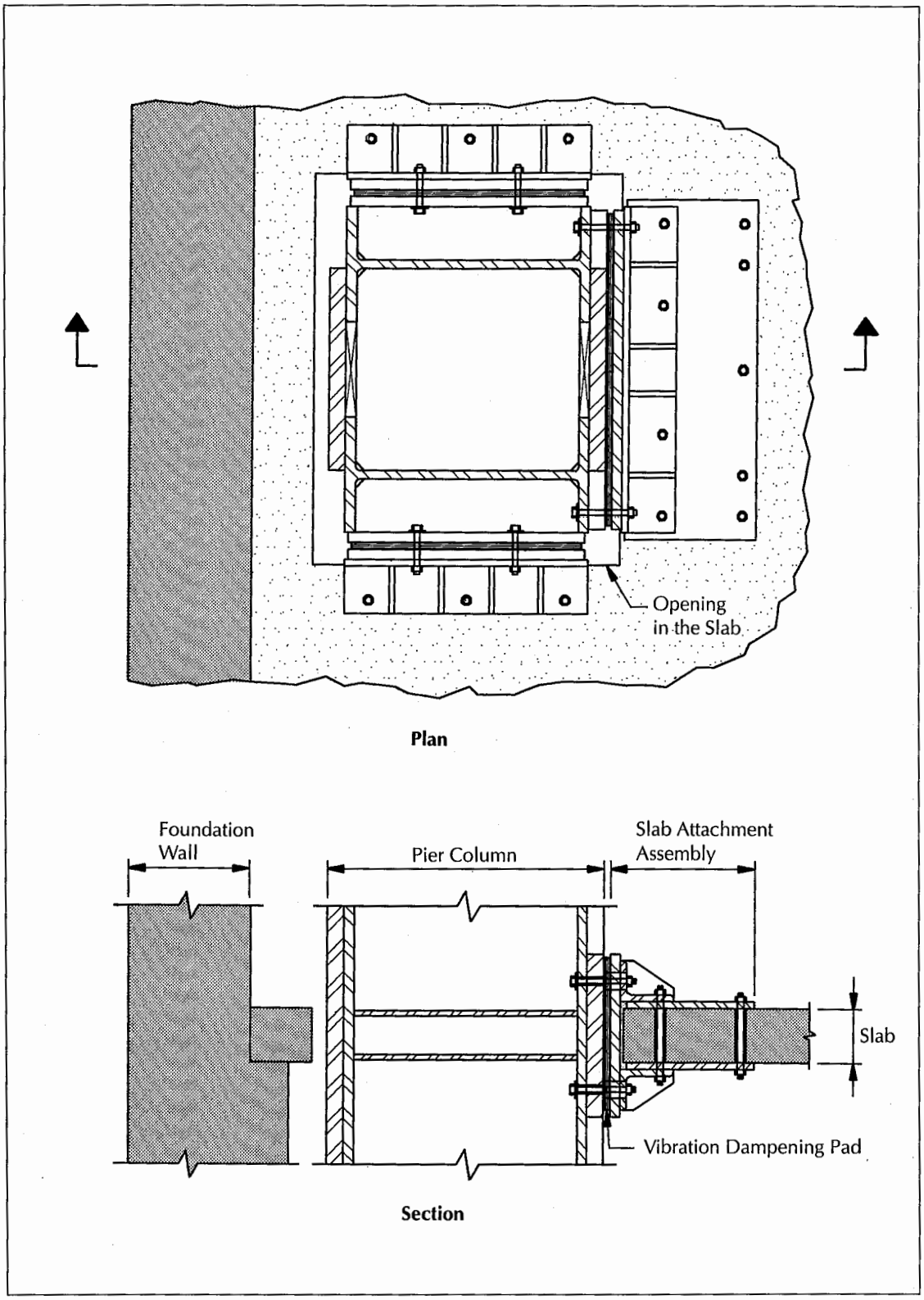


FIGURE 8. Typical steel pier lower support system against the parking level slab.

acceptable. The conventional design approach and common practice called for the separation of the ramp piers from the building structure, but the magnitude of the design loads and the physical restrictions inside the One International Place garage demanded that the ramp be supported laterally by the building.

A dynamic analysis of the ramp structure combined with the existing building was also impractical due to the complex geometry involved and the resulting highly complex computer model that would have been required. The dynamic analysis was therefore focused on the pier structure with rigid supports provided by the building. Different mode shapes and the natural frequency of this simpler model were determined in order to select the appropriate vibration system at the location of the piers' supports. The results of the analysis showed that the low frequency vibrations from the second mode shape was the most likely to excite the existing building structure. Using this information, a vibration dampening system was designed with vibration dampening pads (see Figure 8).

Shock and vibration absorbing bearing pads, washers and sleeves were used in addition to bearing pads in order to completely isolate the pier elements from the garage structure. Even though the design of the vibration isolation system was the result of intense effort, its actual behavior in the field could not be fully predicted by the theoretical models. It was only after the opening of the ramp that the full dampening effects of the isolation system became clear, and since its opening in April of 1990 there has been no report of ramp traffic-induced vibration in the One International Place garage or office building.

Construction Considerations

The last ramp pier inside the garage structure required two supporting columns rather than one — as had been the case for the other piers. The necessity for two columns in this pier was the result of a very long cantilevered box girder (see Figure 9). The dead load of the ramp structure created compressive forces in both box columns, but the live loads on the ramp created a net tension force in the rear box column. The design criteria set by the MHD did not permit the use of tension piles for the dead load of the ramp structure. Only live load tension forces were allowed to be carried by the tension piles. Therefore, in order to avoid the transmission of dead load forces into the tension piles, the following construction sequence was recommended to the contractor:

1. Install the tension piles and anchor into the bedrock.
2. Install the combined concrete footing for both pier columns.
3. Install the forward pier column and tighten all anchor bolt nuts.
4. Install the rear pier column but leave the anchor bolt nuts one inch above the top of the base plate to allow for vertical movement of the column.
5. Complete the construction of the ramp structure at street level.
6. After all dead loads are in place, hand tighten and secure the rear pier column's anchor bolt nuts, and grout under the base plate.

Summary & Conclusion

In summary, the relocation of the High Street ramp is notable for the following reasons:

- It is the first transportation project in the Commonwealth of Massachusetts to make use of drilled caissons and tension piles as a means of foundation support.
- The project's unique vibration isolation system prevented the transmission of vibrations from the ramp structure to the One International Place office building and garage.
- All ramp piers were located and designed so that the ramp could remain operational during the future excavation of the Depressed Central Artery.

It is likely that some of the solutions to the problems encountered during the design of this ramp are relevant to other temporary ramps that are currently under design as part of the depressed Central Artery project.

ACKNOWLEDGMENTS — *The authors wish to thank the following, without whom this project*

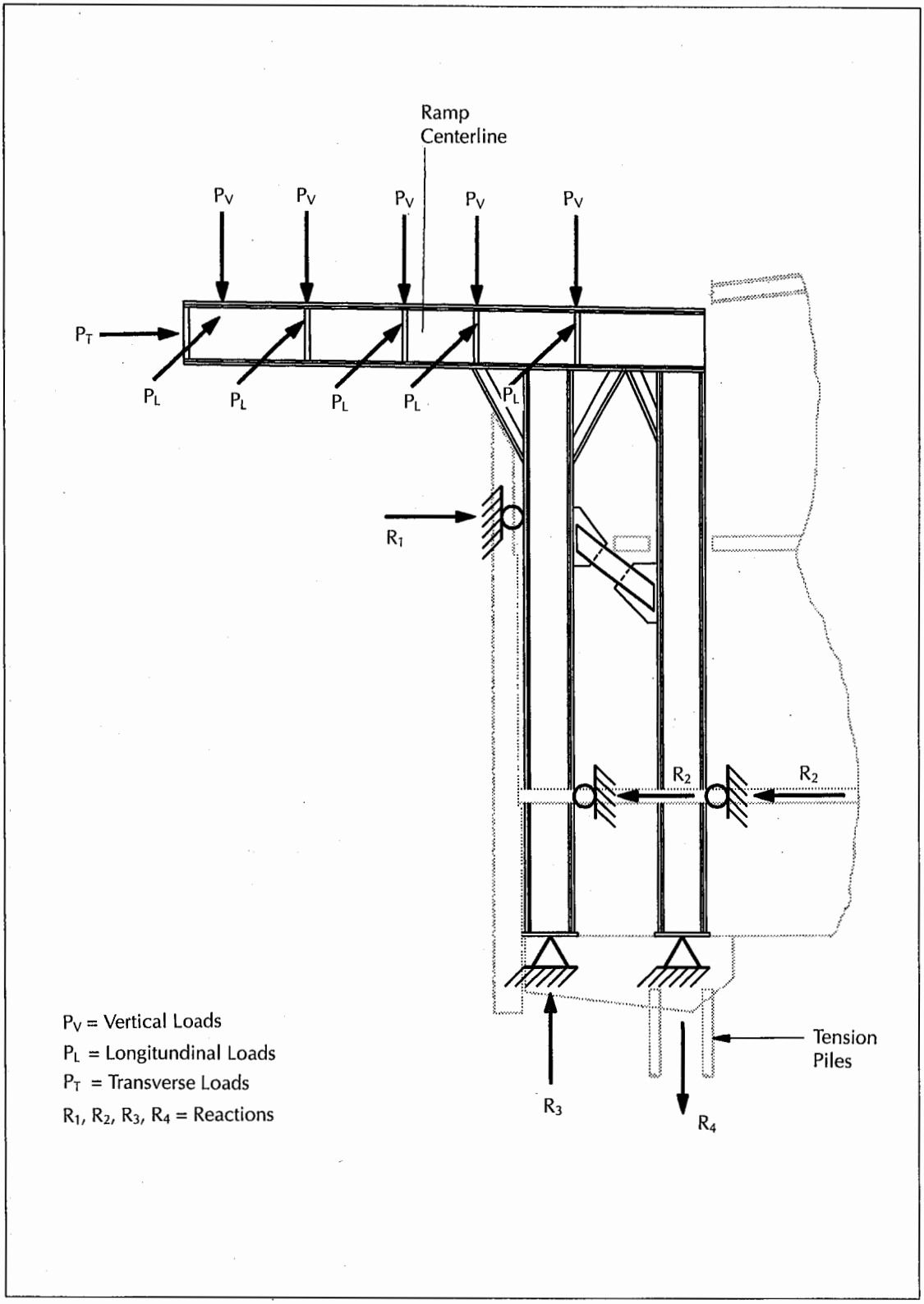


FIGURE 9. Model of the support for the double column steel piers.

would never have been brought to fruition: The Chiofaro Company — Developer of International Place; Massachusetts Highway Department — Owner; Vanasse Hangen Brustlin, Inc. — Civil Engineer; John Burgee Architects — Architect; Haley & Aldrich, Inc. — Geotechnical Engineer; Daniel O'Connell's Sons, Inc. — Contractor; Fabreeka Products Co. — Bearing Pad Manufacturer; and Bechtel/Parsons Brinckerhoff Joint Venture — Reviewers of the ramp design.

ABDOL R. HAGHAYEGHI *is a Principal of Weidlinger Associates, Inc., where he has worked since 1980. He is based in the Cambridge, Massachusetts, office of the firm where he directs the design, management, and construction of an extensive variety of projects. He has earned a B.S.C.E. from Northeastern University (where he is currently working toward his doctorate) and an M.S.S.E. from the Massachusetts Institute of Technology.*

PETER J. QUIGLEY *joined Weidlinger Associates, Inc., in 1987 after completing the B.S.C.E. and M.S.C.E. degrees at the University of Massachusetts in Amherst. In addition to the High Street ramp project, he has taken an active role in the design of office and government buildings, schools, and sports stadiums.*

The Design of a Suspension Bridge Anchorage System

The use of a rock anchorage system with a redundant waterproofing scheme proved to be an economical and safe design solution that accommodated site conditions.

WILLIAM R. HUGHES

The Father Louis Hennepin Suspension Bridge over the Mississippi River in Minneapolis, Minnesota, replaces a steel arch bridge that was constructed in 1889. The steel arch bridge was preceded by two suspension bridges, the first of which was a wood tower suspension bridge that was constructed in 1855. This bridge was the first bridge to span the Mississippi River. It was replaced by a stone tower bridge in 1876. Foundations for the earlier suspension bridges are preserved near the west tower of the new bridge.

The bridge was constructed in longitudinal halves in order to permit uninterrupted traffic movement on Hennepin Avenue during the construction. The south half of the bridge was constructed during 1988-89. The north half was constructed during 1989-90. The main suspension span is 625 feet long, and is flanked by simply supported deck welded plate girder back spans. The overall bridge length — including the main span, back spans, and approach spans — is about 1,050 feet. The bridge was designed to carry six lanes of AASHTO Standard HS25 Truck and Lane Loading, with bikeways and wide sidewalks as shown in Figure 1. The cost for the suspension bridge was approximately $20 million, plus an additional amount of approximately $6 million for the approaches.

Bridge Layout

The design of the Hennepin Suspension Bridge called for a state-of-the-art anchorage system that would restrain a large tension force from the suspension cables. Since the bridge was to be constructed in two longitudinal halves, a total of four suspension cables were required that, in turn, required eight cable anchorages. The center two cables were located seven feet apart. Thus, the interior anchor systems were designed to handle two cables; the exterior were designed to handle one. The three critical elements of this anchorage system, which is designed to economically restrain a tension force of seven million pounds in a localized area, are:

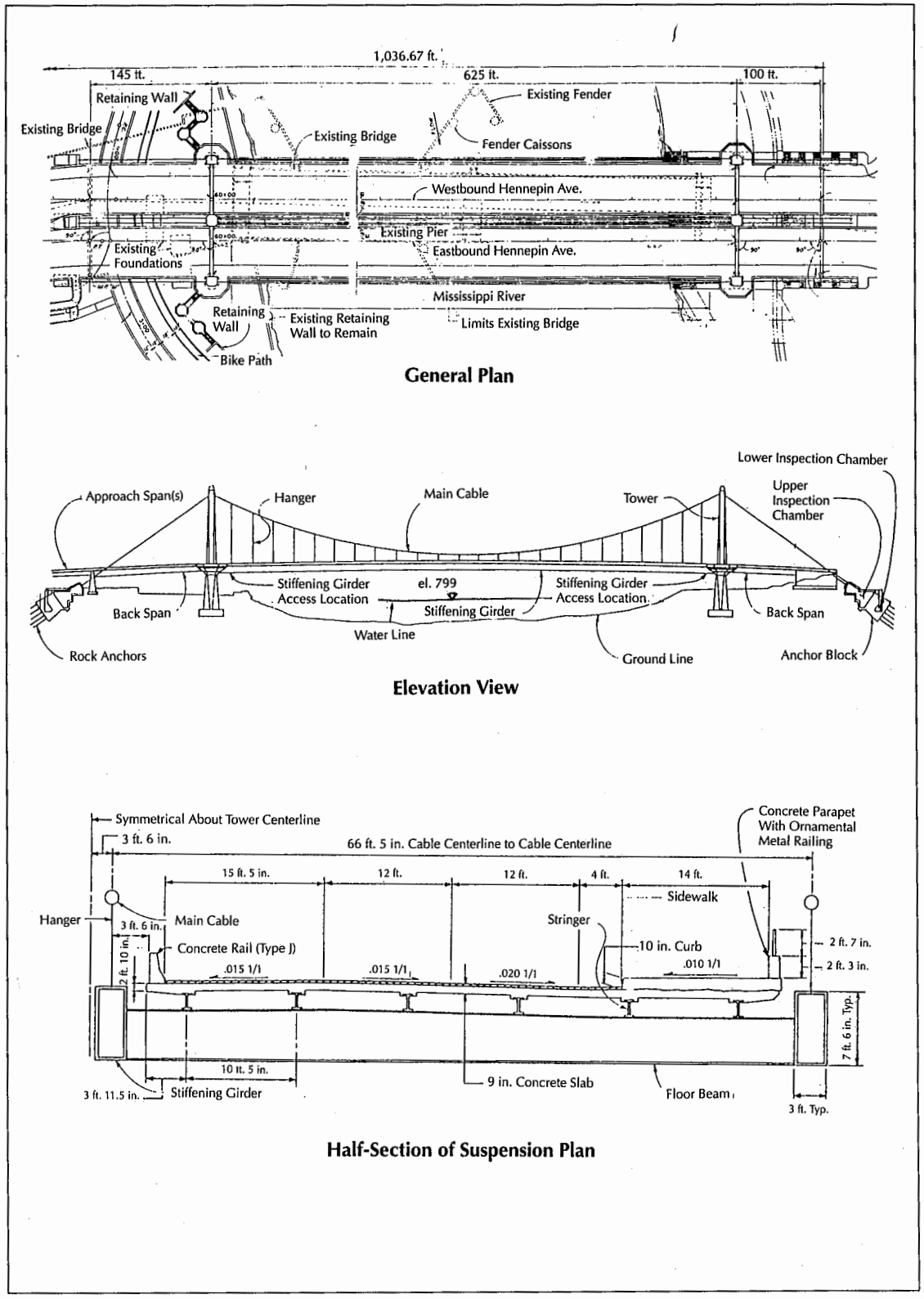


FIGURE 1. Hennepin Bridge plan/elevation.

- The permanent rock anchor tieback system
- The concrete transfer block
- The anchor chamber waterproofing system

Permanent Rock Anchor Tieback Design/Construction

A variety of suspension bridge anchoring systems were evaluated for this project. Among them were gravity anchors, prestressed rock anchors, drilled shafts and a self-anchored system. This study encompassed cost, constructability, long service life, risk and aesthetic factors for each of the proposed systems. Site geometrics eliminated the self-anchored bridge alternative. The use of groups of 30-inch diameter drilled shafts located at several different angles of inclination was investigated, but it was dismissed due to constructability problems, cost and the lack of redundancy.

The final selection came down to a comparison between two alternatives: the gravity mass system and the rock anchor system. The mass anchor system provided the lowest design risk factor due to major dependence on the physical mass of the anchor and the relative predictability of the geotechnical bearing parameters. However, the large amounts of rock excavation and concrete placement that would have been required resulted in a high cost for this option.

The proximity of the sandstone bedrock resulted in making the prestressed rock anchors a viable choice. Potential concerns with the rock anchor system were as follows:

- Permanent prestressed rock anchor tiebacks are a relatively new technique to the United States. They were first installed in the 1960s in any significant quantity. Anchors have been used in Europe prior to that, however.
- The corrosion protection system has the potential to create problems. Failures have been reported, largely due to corrosion in the vicinity behind the anchor head, in aggressive environments.
- Potential long-term creep of the sandstone bedrock would reduce anchor capacity. Other anchor losses (predicted to be fairly small) could also mean that the anchors would loose additional capacity over time.
- Confined space limitations meant that each rock anchor tendon would need a design capacity of 700 kips, which is quite large in comparison with other permanent anchors installed to date.
- The failure mode of a "wedge" or "bulb" pullout of the sandstone bedrock would need to be evaluated.
- Many rock anchor systems are proprietary and it is difficult to specify a system with the latest technology for a publicly funded project.
- The bond strength (or shear strength) between the grout and various rock types are different. Even for sandstone, available guidelines give a range. What value should be used? What will the safety factor be?

Cost estimates showed the rock anchor system would be the economical method to transfer the suspension cable tension force to the sandstone bedrock. The designers concluded that if the above concerns could be suitably addressed, this method would be selected. The overall concept is shown in Figure 2. The suspension cable locks off to a metal plate bearing against a concrete anchor block that is restrained to the sandstone by prestressed permanent rock anchors. The anchor block and chambers can be more easily visualized by reviewing the anchor isometric shown in Figure 3.

Even though the use of permanent "prestressed" rock anchors is relatively new to the United States, its longer track record in Europe provides a significant body of experience. A number of reputable rock anchor contractors have been successfully installing rock anchors in the United States. The Post-Tensioning Institute (PTI) has recently developed its "Recommendations for Prestressed Rock and Soil Anchors." The project's designer decided to require that the anchors be installed per the PTI recommendations. The designer also stipulated that the general contractor engage a specialty rock anchor foundation subcontractor that has had at least ten years of successful

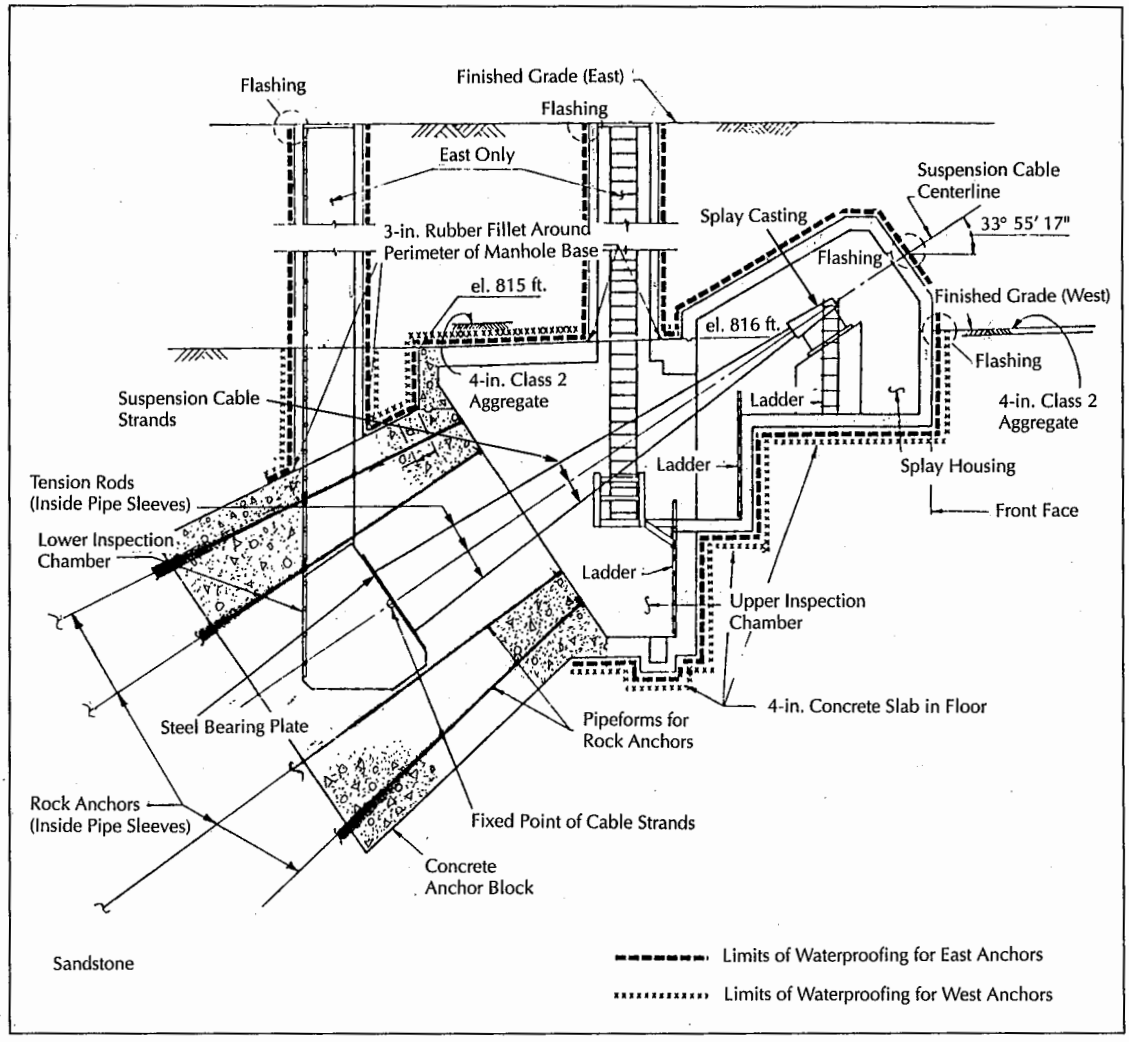


FIGURE 2. Typical section through the anchor.

experience installing similar high-capacity anchors. Competitive bids were received by general contractors.

Because the longest life possible was desired for the anchorages, the designer decided to require a dual corrosion protection system, which essentially is a grouted tendon encased in a corrugated polypropylene sheath that, again, is surrounded by grout (see Figures 4, 5 and 6). The grout and the sheath provide the double protection. The ground environment was evaluated and determined to be non-aggressive in regard to corrosion. In addition, the anchor head will be protected from the weather. The head is housed inside a humidity-controlled inspection chamber and is covered with a removable grease-filled cap.

The PTI recommendations require an extensive testing program as well as acceptance criteria for jacking (stressing) the anchors. Performance, proof and seven-day lift-off tests were required to ensure that the anchors were properly tensioned. The design incorporated additional (spare) locations for anchors that would not be used unless failures were encountered (anchors not meeting acceptance criteria). The anchor head was designed so that it could be jacked in the future (without unseating the wedges) to verify the anchor force. At the same time, metal shims could be inserted

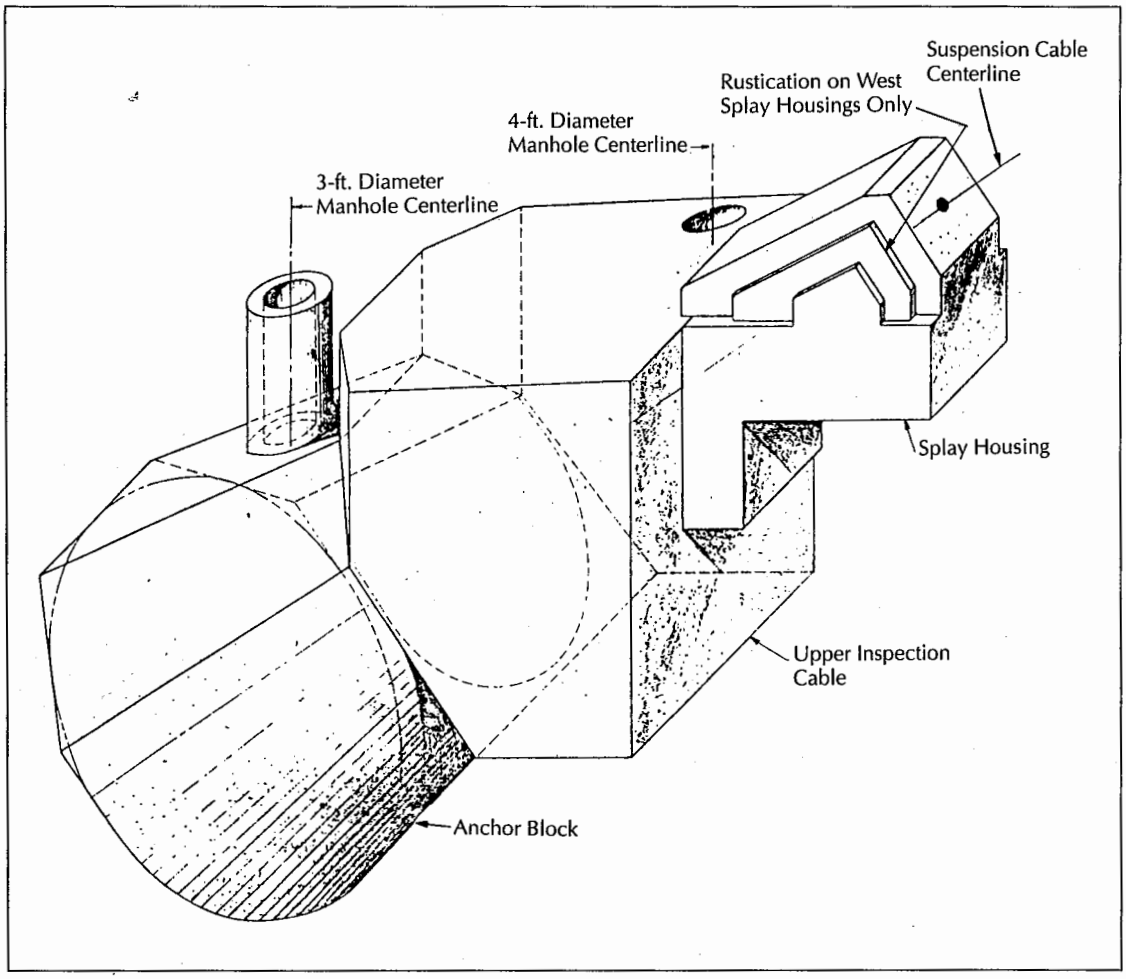


FIGURE 3. Anchorage isometric.

behind the head to increase the anchor force, if necessary.

The anchors were fanned out radially (see Figures 7 and 8) and the bond lengths were staggered to engage a large volume of sandstone bedrock in order to increase the safety factor against a bulb (or wedge) failure.

The anchor system was designed and specified as a generic system that could incorporate a number of desired features of several proprietary systems available in the marketplace. Any system proposed by the contractor would need to satisfy the required features: either strands or bars would be acceptable.

A pullout test was specified to be performed by the rock anchor contractor in order to determine the ultimate shear strength of grout to sandstone. A length of approximately five feet was grouted in sandstone and tested by jacking. The results were somewhat inconclusive due to construction variables. However, the project engineer was confident that the design shear strength was achieved.

The design rock anchor load was approximately 700 kips, with total anchor lengths ranging from 80 to 105 feet. Each tendon consisted of nineteen 0.6-inch diameter seven-wire strands inside a corrugated polypropylene sheath. The tendon was grouted along its full length, including the stressing length (inside and outside of the sheath) in one step so there would be no cold joints that would possibly lead to a breach of the dual corrosion protection system. The unbonded (stressing) length consisted of grease-injected strands encased in plastic jackets.

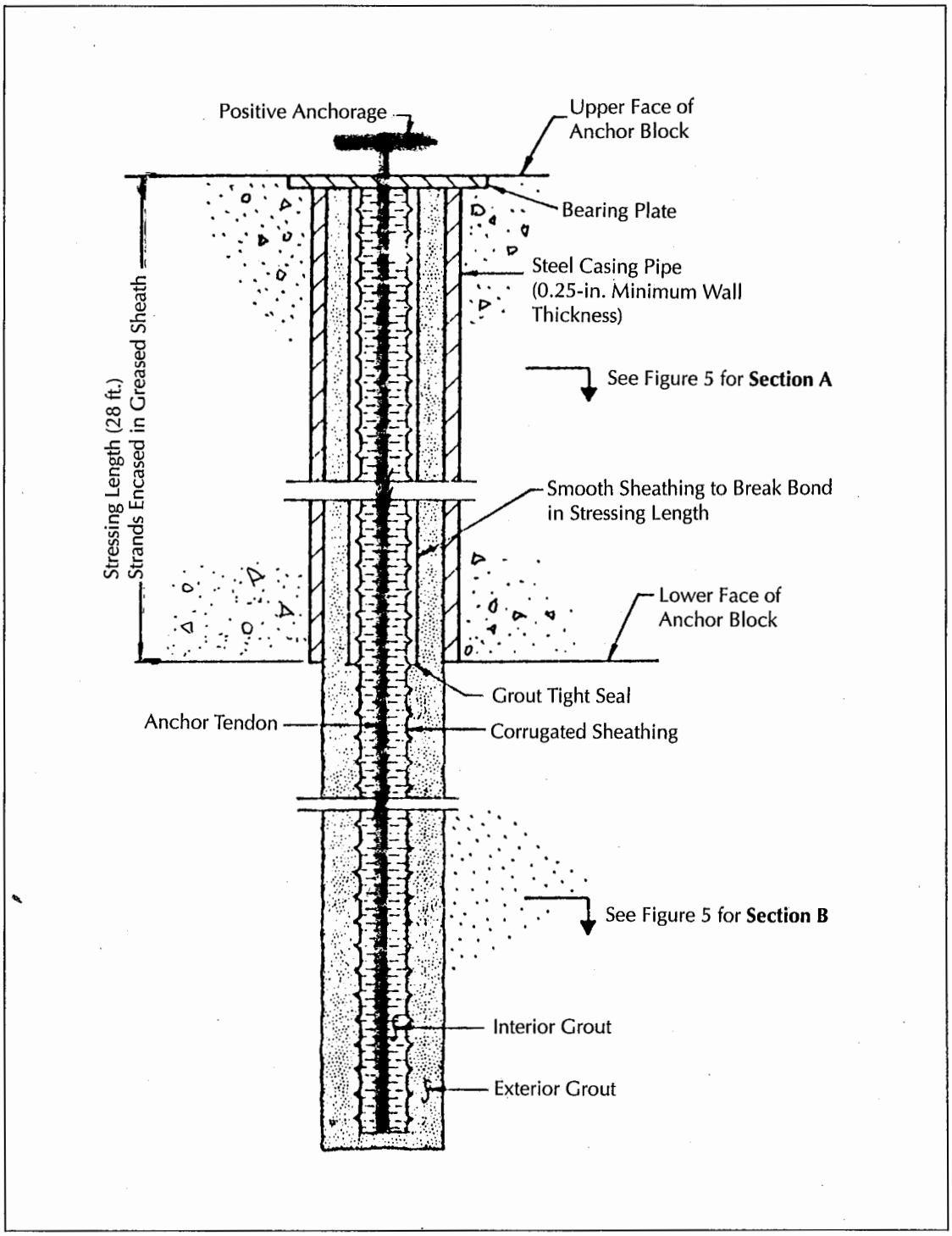
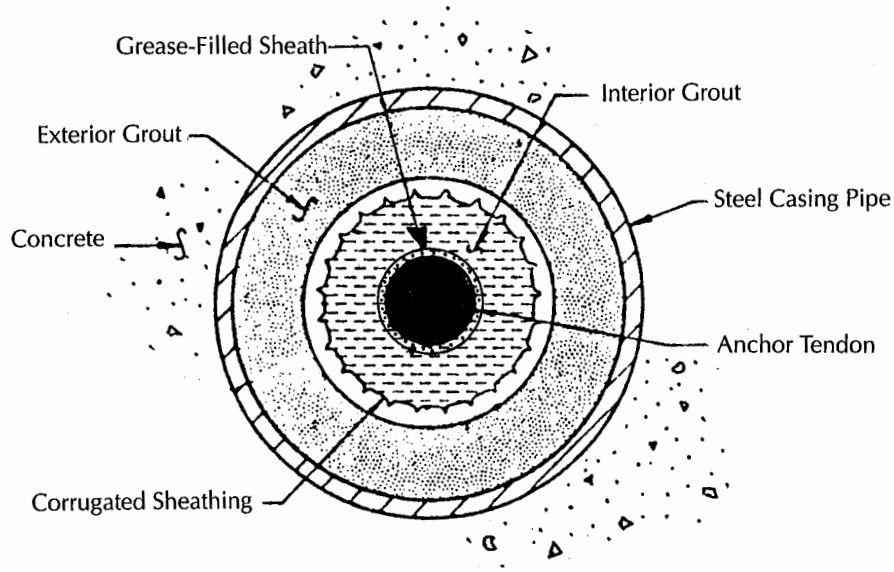
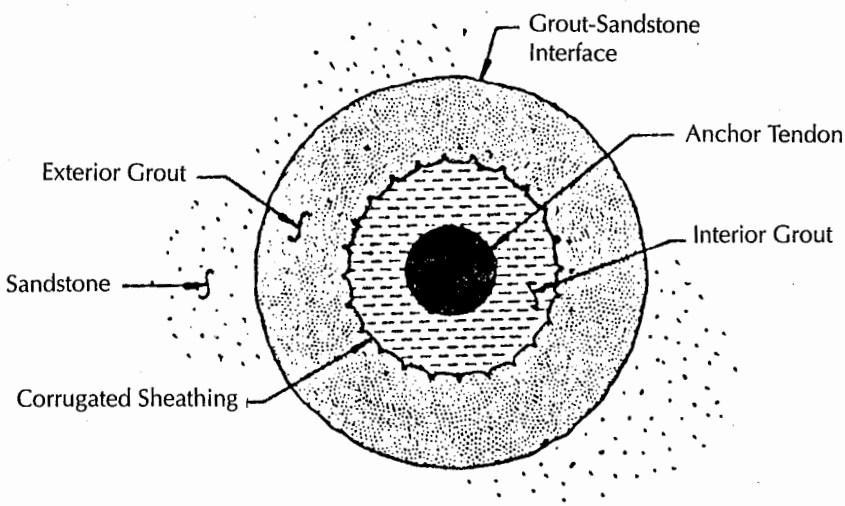


FIGURE 4. Rock anchor detail.

The drilled holes required for the strands/sheath (tendon) were seven inches in diameter. Holes were drilled with standard drill rigs and cuttings were flushed to the surface with water. Careful drilling and flushing procedures were implemented in order to pre-



Section A (Stressing Length) from Figure 4



Section B (Bond Length) from Figure 4

Note: No scale.

FIGURE 5. Rock anchor sections.

vent wall sloughing and caving. Grout (with no additives) was pumped to the bottom of each hole in a plastic tube where it could rise gradually and simultaneously inside and outside the corrugated sheath. It was necessary to install the corrugated sheath at the site in order to

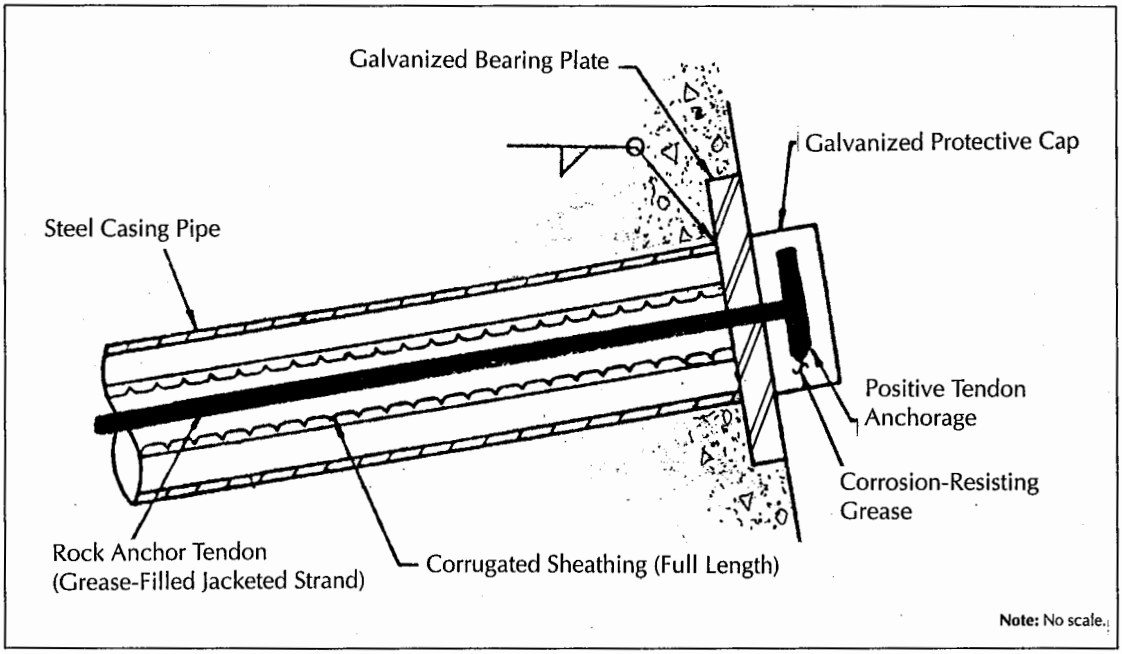


FIGURE 6. Rock anchor head detail.

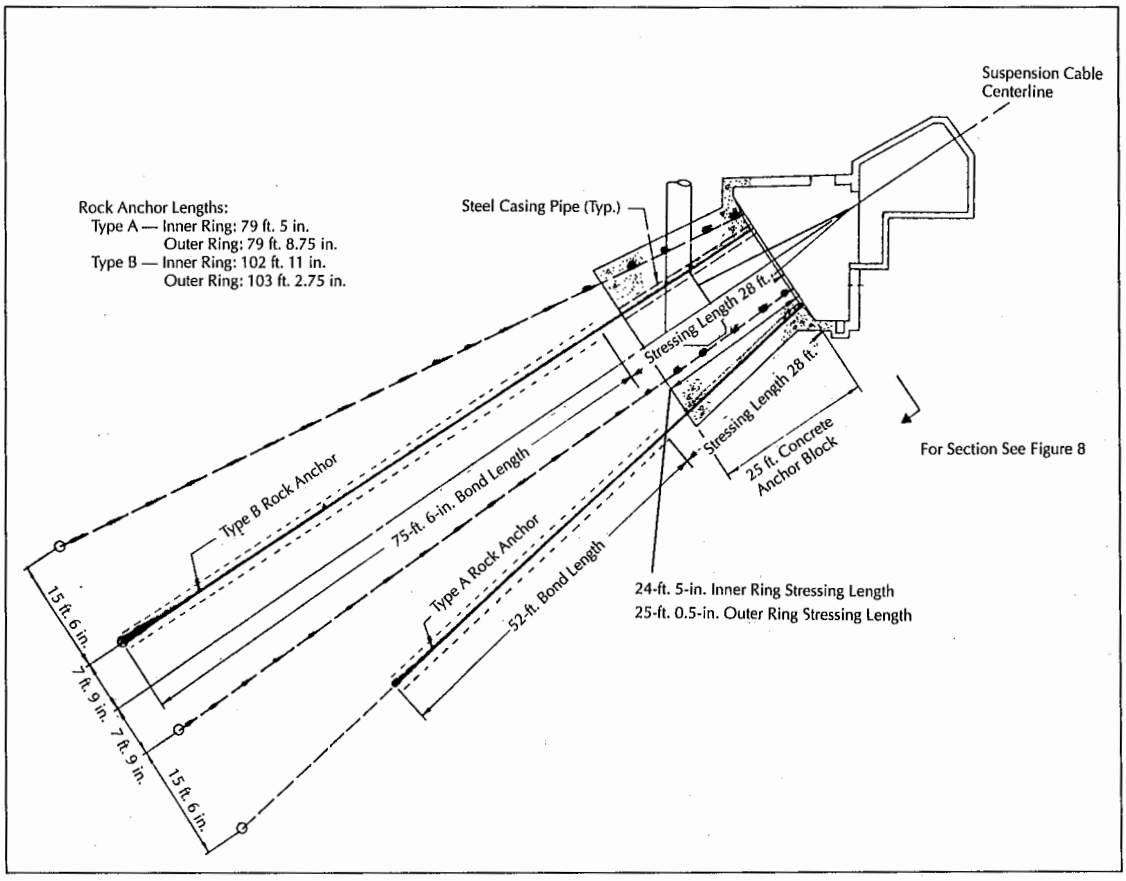


FIGURE 7. Rock anchor layout.

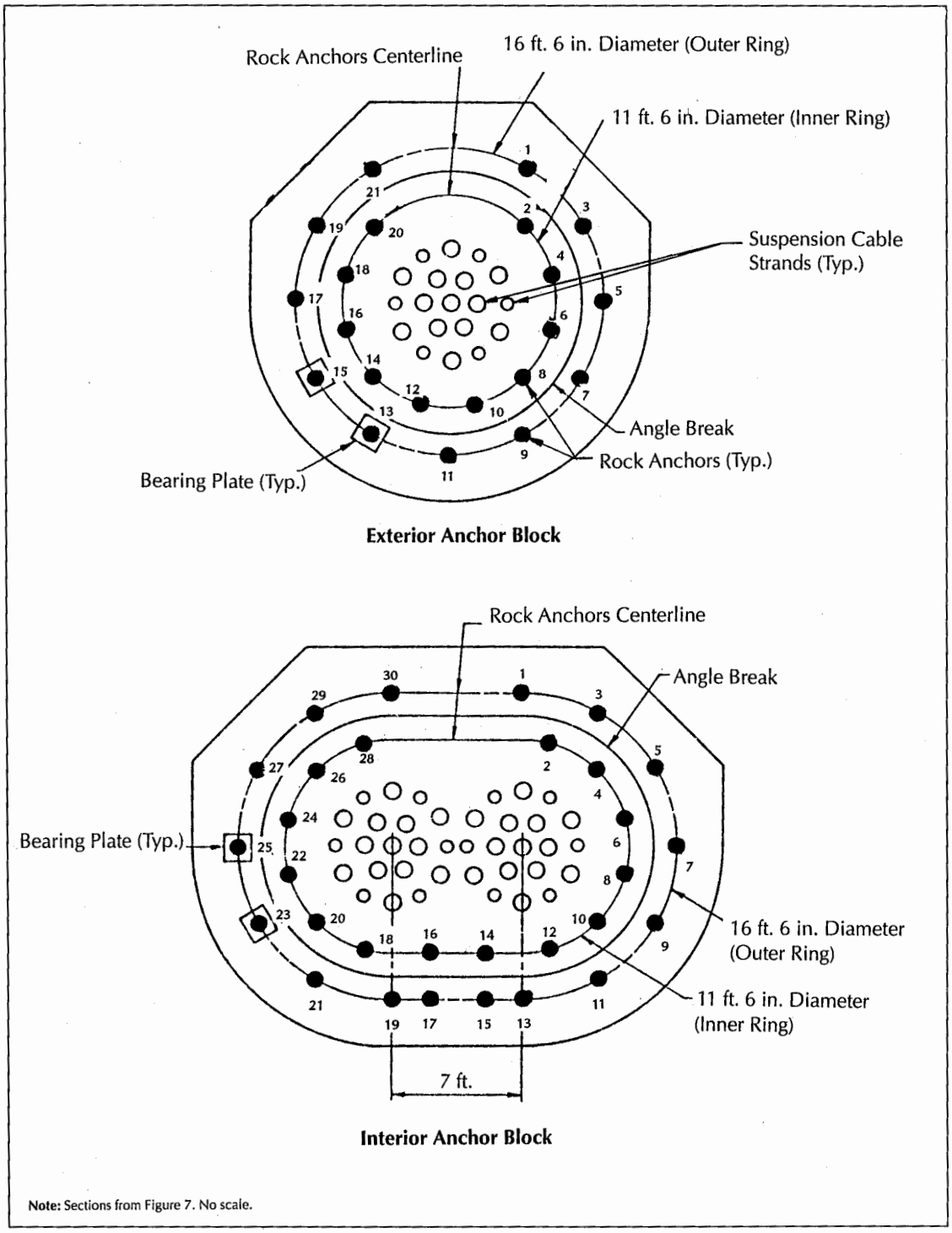


FIGURE 8. Section of interior and exterior anchor blocks.

avoid damage from shipping and coiling. Standard hydraulic rams were used for jacking. The unit bid price for installation and stressing was $9,000 each for the shorter tendons and $10,000 each for the longer tendons, inclusive of drilling.

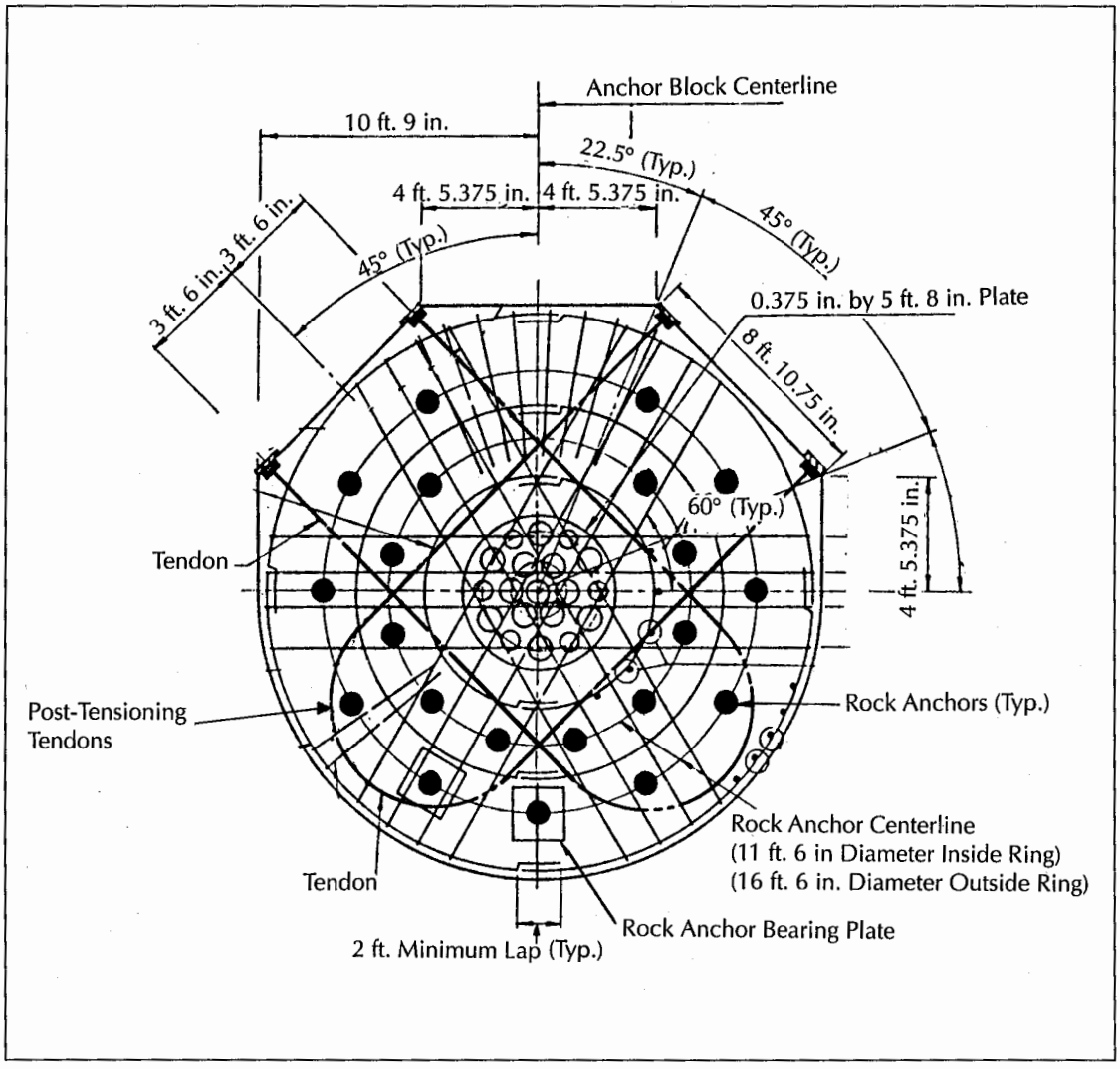


FIGURE 9. Top face of an exterior anchor block.

Concrete Transfer Block Design/Construction

A structural element was required to transmit the tension load from the suspension cable to the rock anchors. The 19 strands of the suspension cable fanned out radially to a diameter of six feet. The strands were socketed and attached to a steel threaded rod that was inserted in steel sleeves through the concrete anchor block, and locked off against a steel bearing plate with a large nut (see Figure 2). The rock anchors were located around the perimeter of this plate. Therefore, an element that would transfer the load from the bearing plate to the perimeter rock anchors was required. Metal pipe sleeves were provided through the concrete anchor block for both the suspension cable tension rods and the rock anchors in order to facilitate placement and alignment.

A cylindrical concrete transfer block was designed (see Figure 2). Potential failure modes of this block included shear (pullout) of the metal bearing plate and bursting from potentially large compressive stresses.

For the shear (pullout) stresses, the shear capacity of the concrete was supplemented with shear friction reinforcement designed per the AASHTO specifications. Possible lo-

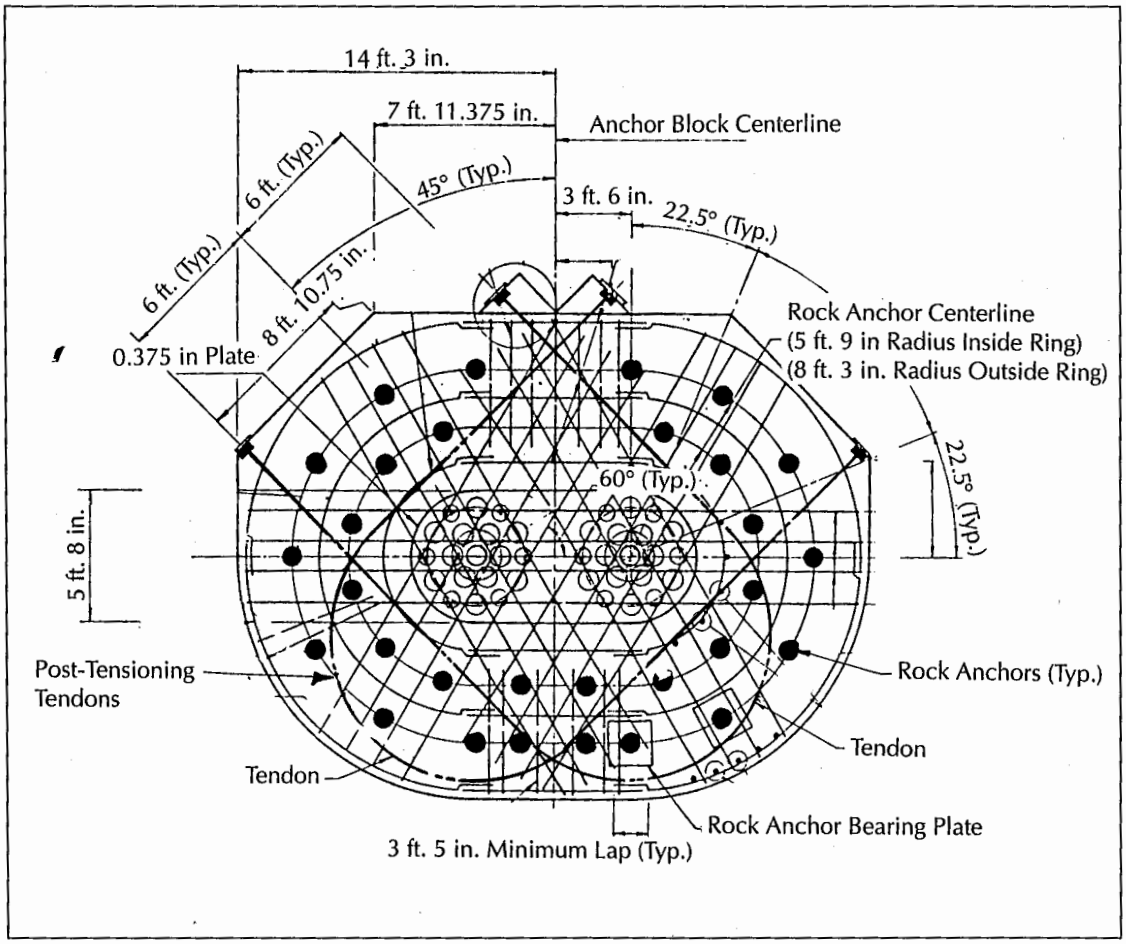


FIGURE 10. Top face of an interior anchor block.

cations for this reinforcement were identified and clearances for concrete placement were checked.

For the bursting (or splitting) stresses, mild steel reinforcement hoops were provided around the perimeter of the cylinder in addition to radial looped post-tensioning (see Figures 9 and 10). The designer's intent was to reinforce the concrete in order to counteract the bursting stresses with a combination of a passive reinforcing bar system and an active post-tensioning system, and to prevent any concrete cracking that could lead to water intrusion and corrosion. The calculation of these stresses was based on the same theory that was used to determine the bursting stresses behind a post-tensioning anchor head.

To determine the tensile (splitting) forces behind an anchor head, the equation is:

$$Z = 0.3P\,(1-a/d)$$

Where:

Z = the total splitting or bursting force,
P = the tendon force,
a = the depth of the anchor place, and
d = the depth of the concrete section.

The development of this equation is discussed in Leonhardt's "Prestressed Concrete Design and Construction."[1]

Either spiral (hoops) or mats of straight reinforcement can be provided directly behind the anchor head to handle the stresses. Because of the potentially large splitting force, the limited space for mild reinforcement and the desire to provide an active system, the radial post-tensioning system was designed and detailed based on the above splitting force equation that

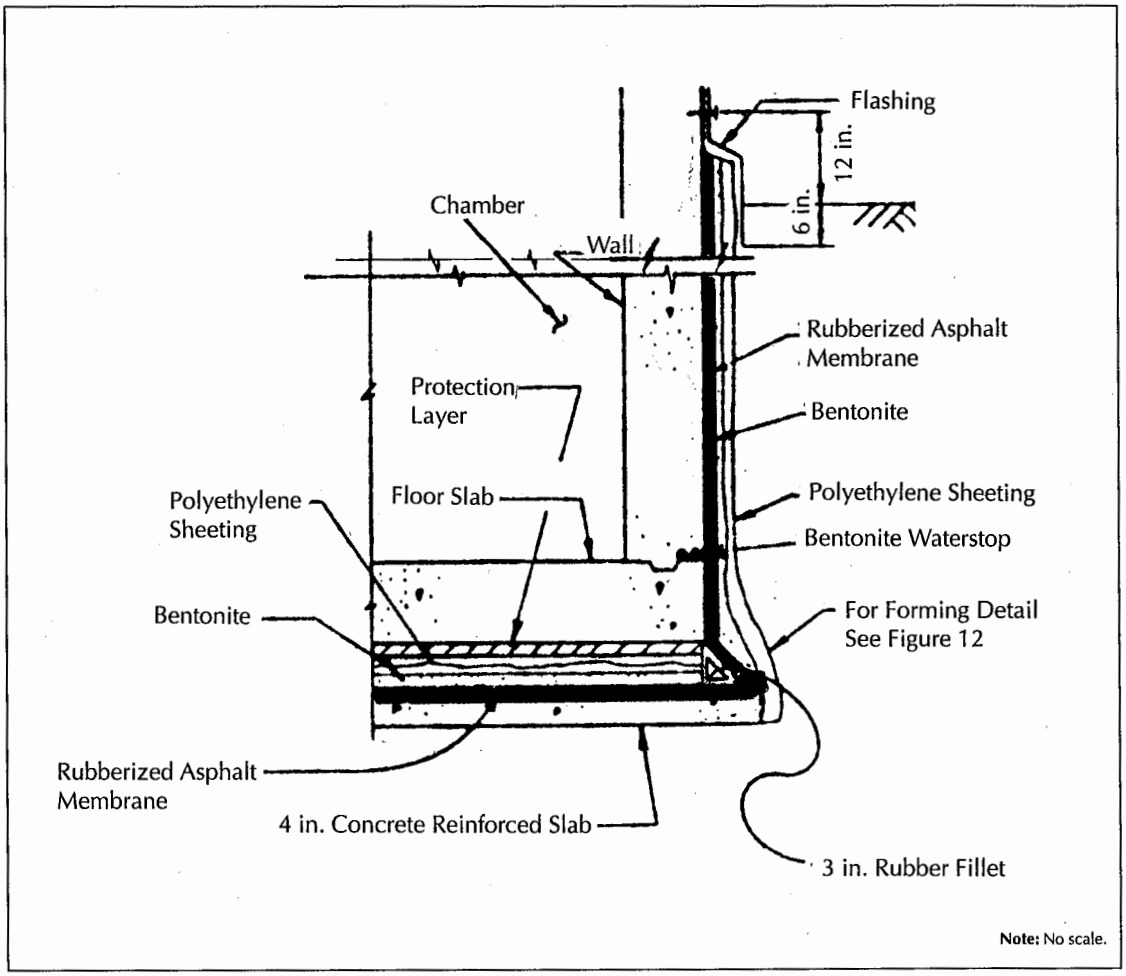


FIGURE 11. A corner detail for the anchor chamber waterproofing system.

was modified in order to reflect a massive section.

The mild steel reinforcement was installed by the general contractor along with the radial post-tensioning hardware. The concrete design strength was 4,500 psi.

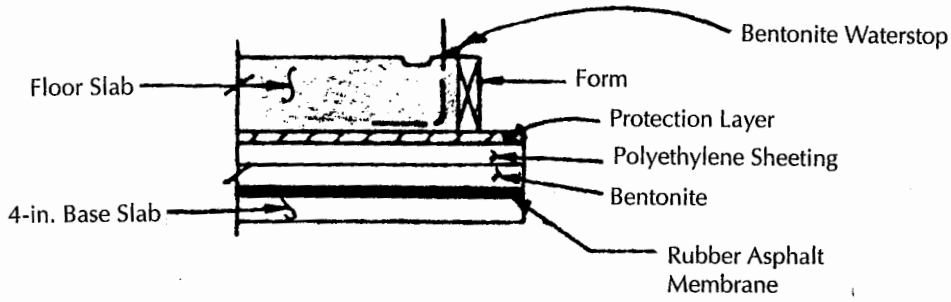
Unit bid prices were $0.45 per pound for the mild reinforcement, $225 per cubic yard for the concrete and $9,000 for each anchorage's radial post-tensioning system.

Anchor Chamber Waterproofing Design/Construction

Buried chambers (or vaults) were required for suspension cable installation and for future inspection purposes. These chambers are located below the water table and are buried under the approach roadway with up to fifteen feet of fill placed above them (see Figure 2). There was a need to provide a dry environment for long-term corrosion protection of the suspension cable and related hardware. Because of that requirement and because the waterproofing system would not have access to repair any leak, the designer decided to provide for two independent waterproofing systems.

After studying the desirable features of numerous available systems, the designer decided to combine the two following systems for this particular installation:

- A layer of bonded rubberized asphalt membrane that was glued to the exterior of the concrete chamber walls.
- A layer of sprayed bentonite that was covered by a polyethylene sheet and a protec-



Note: A 1.5-in. chamfer is placed on corners in contact with waterproofing. No scale.

FIGURE 12. Waterproofing floor slab forming detail.

tion board in order to protect the systems against damage from the backfilling operation.

While the rubberized membrane is a positive impermeable sheet of uniform thickness, it does have seams that can leak. It also does not lend itself to accommodating irregular shapes and corners.

The bentonite system is sprayed on so there are no seams. Also, it covers irregular shapes well. Nevertheless, the presence of salts (which could form in the future) can affect its swelling properties and, therefore, detract from its waterproofing ability.

Both systems can present problems if constructed using poor workmanship. The intent of providing dual systems was to prevent moisture infiltration in a submerged condition, assuming worst case scenarios.

Again, specifications were developed on a generic basis with these features required. Special details were devised for the floor and corner terminations and detailed flashing requirements were specified (see Figures 11 and 12). Manufacturers and applicators were required to provide shop drawings of all corner and termination details. The specifications mandated that the work to be completed by experienced applicators under the direction of the general contractor and of the manufacturer's representatives, as well as under the constant inspection of the project's engineer.

A de-humidification system was also provided within each chamber to keep the relative humidity at or below 35 percent, at which point the rate of steel corrosion slows to a low rate. The chambers were relatively small, so no active method of air circulation was felt to be needed. Open pipe sleeves connect the lower inspection chamber to the upper inspection chamber in order to allow passive air circulation.

The unit bid price for the entire waterproofing system was $2.65 per square foot.

Conclusion

A thorough field inspection program was put in place to carefully monitor and record all of the contractors' work regarding the construction of the anchorage system components. A maintenance manual was prepared to guide future engineers in monitoring the performance of these anchorages. The Hennepin Avenue Suspension Bridge is now complete and the anchorage system, as well as the rest of the structure, is functioning as intended and serving as a landmark structure in the historical district of Minneapolis.

ACKNOWLEDGMENTS — *The new suspension bridge was designed by Howard Needles Tammen & Bergendoff for the Hennepin County Department of Public Works. For oversight and review, staff at the Minnesota Department of Transportation, in addition to the FHWA, supplemented county personnel. The general contractor, Johnson Brothers Corp., Litchfield, Minnesota, engaged nationally experi-*

enced Nicholson Construction Company of Bridgeville, Pennsylvania, as the specialty rock anchor subcontractor. The radial post-tensioning hardware was supplied by VSL Corporation.

WILLIAM R. HUGHES *graduated with a B.S.C.E. from the University of Minnesota after working as a construction inspector for the Hennepin County and Minnesota departments of transportation. As a project engineer with HNTB, he served as designer of the Hennepin Suspension Bridge anchorage system and tower structures. During the construction, Hughes served as HNTB's Resident Engineer to augment Hennepin County's on-site engineering staff. He is currently Director of Engineering in HNTB's office in Irvine, California, where he manages large bridge projects in California.*

REFERENCE

1. Leonhardt, F., "Prestressed Concrete Design and Construction," William Ernst & Son, Berlin, 1964.

Electronic Toll Collection & Traffic Management in Italy

An effective telecommunications system is a key to the successful implementation and operation of electronic toll collection and traffic flow management systems.

JOHN COLLURA

With electronic toll collection and traffic management (ETTM) projects in operation or underway in Oklahoma, Texas, California, New York, Massachusetts and other states, the interest in the use of advanced technology in transportation continues to grow in the United States. A review of the ETTM effort of Autostrade S.P.A., Italy's largest toll road agency, as well as a discussion of future ETTM activities being planned by the Autostrade may be useful for the implementation of current, and the design of future, ETTM projects in this country.

Background

The Autostrade is a quasi-governmental agency that operates 3,000 kilometers of toll roadway. Figure 1 shows the extent of the Autostrade road network, along with other associated road systems. It is owned by stockholders, but its rates (tolls) are regulated by the government. One of many toll road agencies in Italy, the Autostrade also has controlling interest in other toll road agencies in the country.

The Autostrade has been using advanced technology for a variety of ETTM purposes. For more than 15 years, Autostrade engineers, computer scientists and telecommunications experts have worked in a telecommunications laboratory outside of Florence in an aggressive, Da Vinci-like style on developing a comprehensive incident management program. In addition, the Autostrade currently employs both stop and non-stop forms of electronic toll collection using a crediting/debiting method that would have been the envy of former Medici bankers. In fact, the Autostrade's understanding of political and institutional concerns, on the whole, would have impressed even Machiavelli. It is not surprising that this agency's work ethic and operational expertise have translated into successful implementation of advanced technology in order to increase safety and reduce traffic congestion.

Toll Collection

A traveler can pay a toll on the Autostrade in one of three ways:

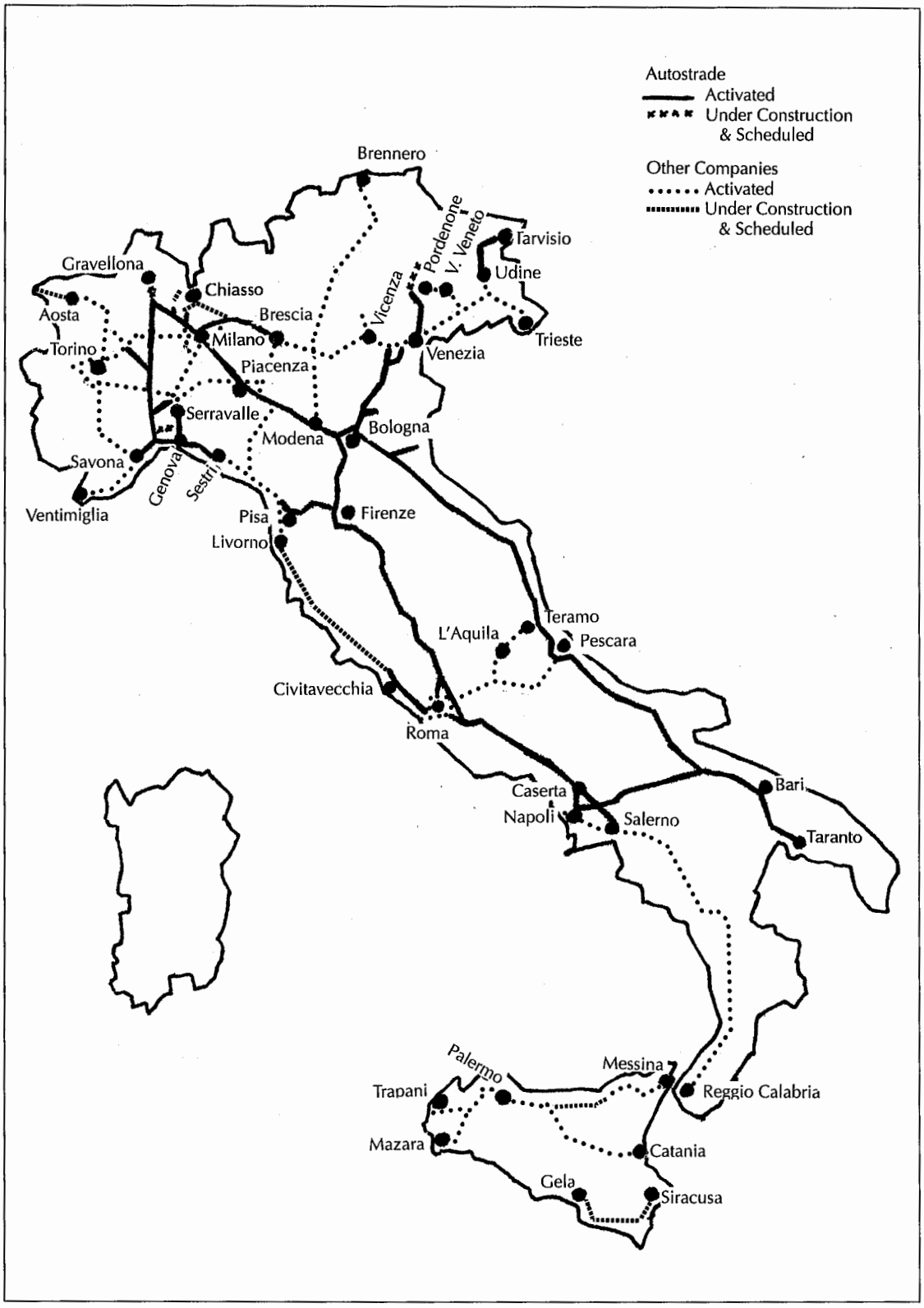


FIGURE 1. A map of the Autostrade network showing the vast coverage of toll roads in Italy.

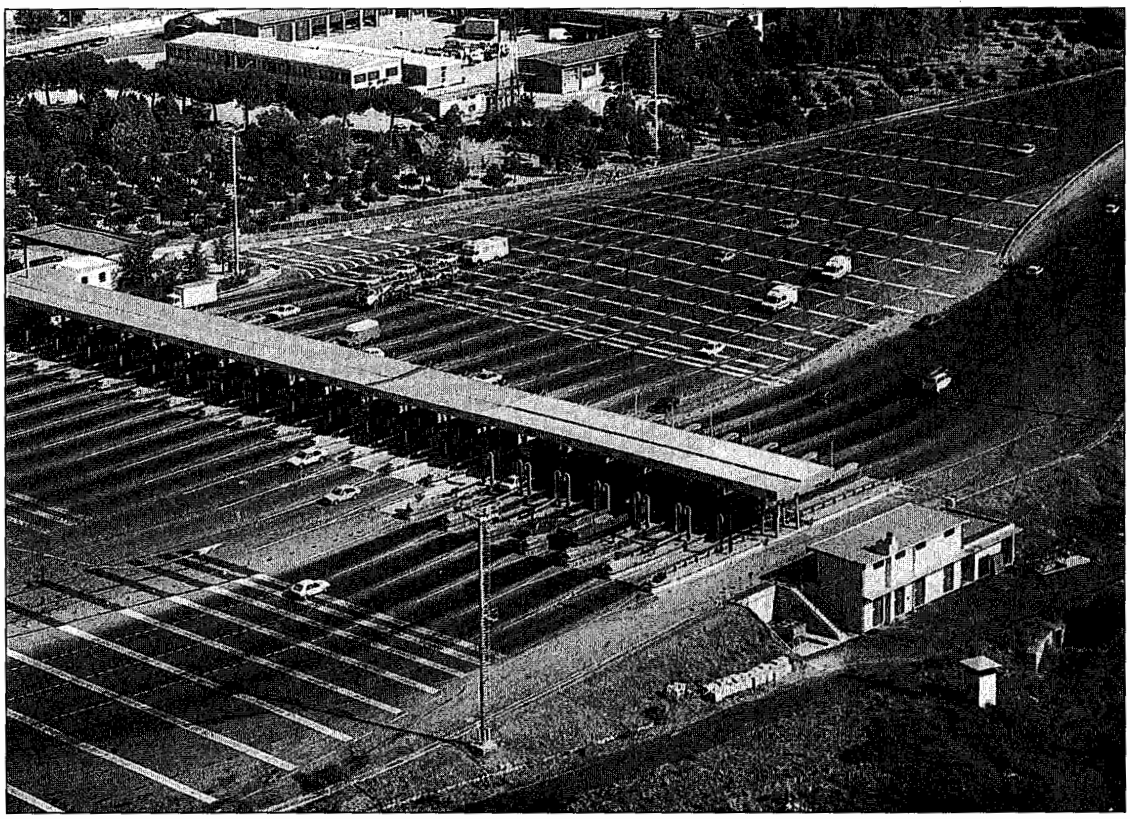


FIGURE 2. A view of a large Autostrade toll plaza.

- *Contante* (with cash);
- With a ViaCard; or,
- With a Telepass.

The ViaCard and Telepass are both used as part of the Autostrade effort to automate toll collection. At present, 35 percent of the tolls on the roads are collected using the ViaCard and Telepass. Along certain corridors, toll plazas accommodate all three methods of collection (see Figure 2).

ViaCards. The Autostrade has used the ViaCard since 1982. The ViaCard is a magnetic plastic card that is used in a stop form of electronic toll collection. Motorists enter clearly marked ViaCard toll gates and stop to insert their ViaCards and toll tickets (where necessary) into the ViaCard computer. The collection system accommodates two types of ViaCard: current account and deduction ViaCards.

With the current account ViaCard, the computer debits the toll fare directly from an account that has been set up with the Autostrade or a participating bank. Motorists maintain about 800,000 active current accounts.

Travelers can purchase deduction ViaCards in denominations of 50,000 to 90,000 *lire* (about $40 to $75) at local tobacconist shops, restaurants and service areas on the highway, as well as at special Autostrade offices. The deduction ViaCard is also inserted into a computer that reduces the value of the deduction card after each toll payment. Approximately three million deduction ViaCards are sold each year.

Telepass. The Telepass, a plastic "smart" card, contains a microchip and is used in non-stop electronic toll collection. The Telepass payment system is similar in set-up to the current account ViaCard. Telepass users may pass through designated Telepass toll gates at up to 30 km/hour (see Figure 3). Speed is limited at that rate for safety purposes only; the system can function with vehicles passing by at speeds up to 130 km/hour. Vehicles are also requested to maintain a safe distance of 20 meters from each other in the lane.

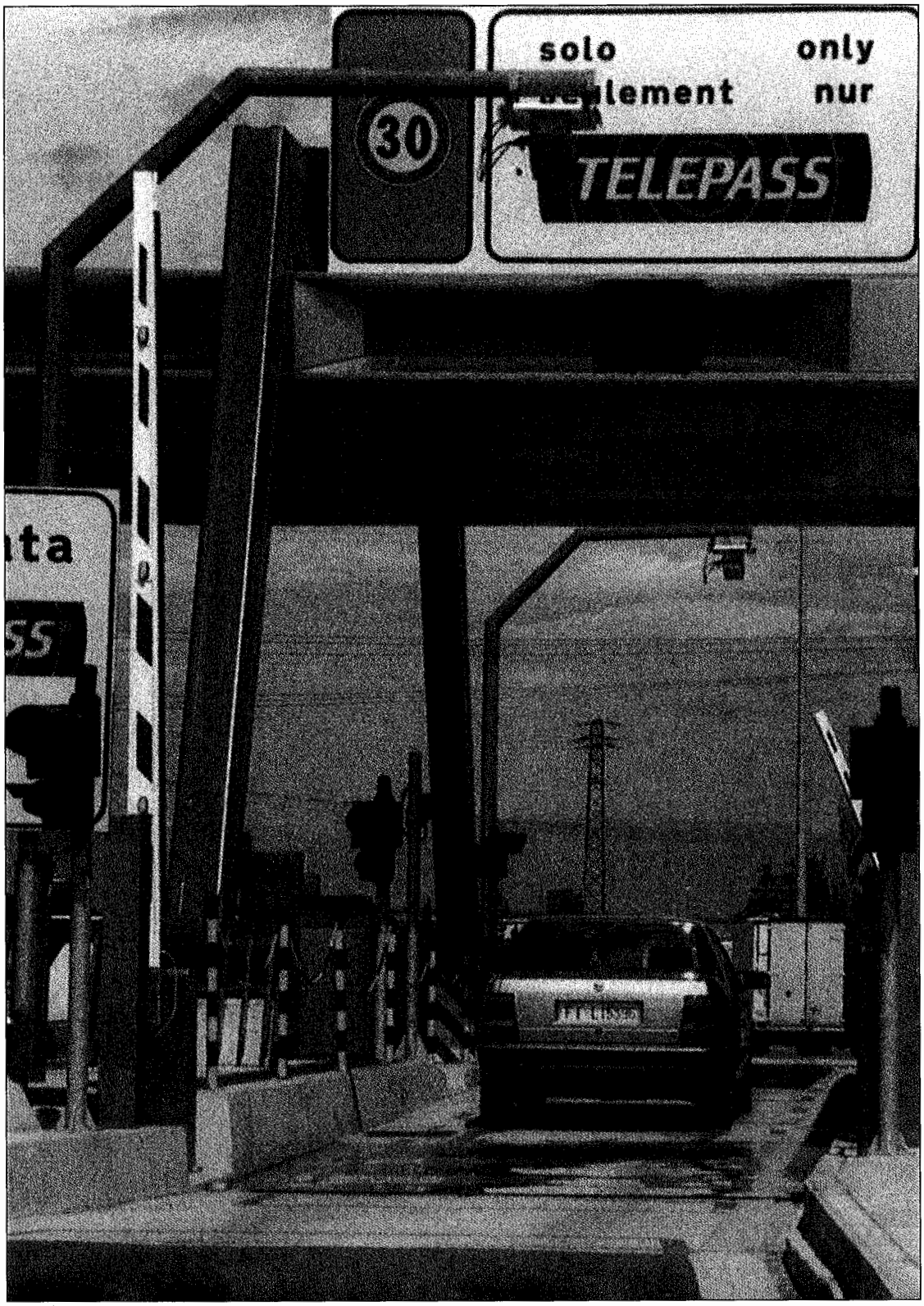


FIGURE 3. A view of a Telepass toll collection lane.

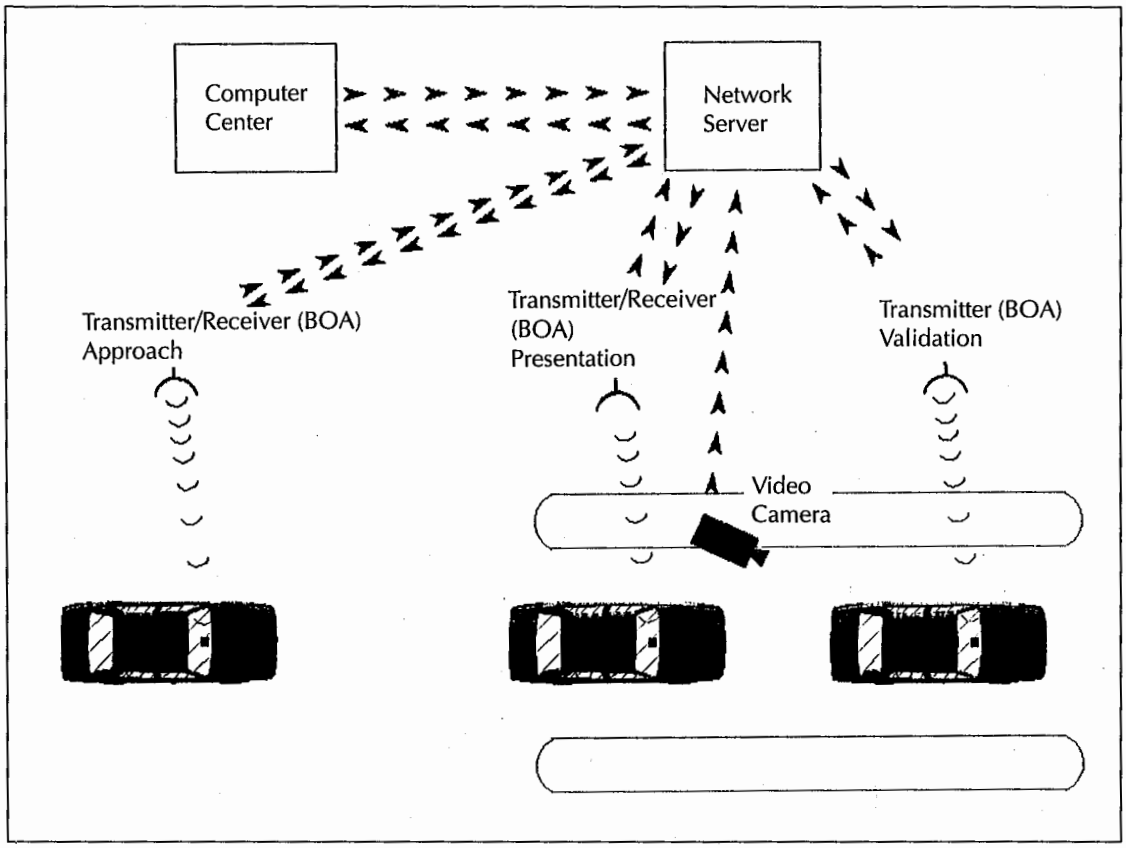


FIGURE 4. A schematic of the Telepass telecommunications system.

Before entering the approximately 500-meter-long Telepass lane, the motorist inserts the Telepass into a two-way communication device (about the size of a cigarette pack) that is mounted on the top center of the windshield. As the vehicle passes, the system reads and writes data on the Telepass via a series of overhead radio frequency (RF) units (see Figure 4). These units (BOA) operate at frequencies from 5,785 to 5,815 gigahertz.

If the system fails to verify the Telepass, perhaps because of an insufficient credit account balance, it transmits audio and visual signals to the vehicle via the mounted communication device while the motorist still has time to switch from the Telepass lane to a manual toll gate. If the motorist ignores the alarm and uses the Telepass lane, a video camera captures an image of the vehicle's license plate. Its registration number is placed on the Autostrade's *lista nera* (blacklist) and further administrative action is taken.

All toll payment transactions are stored on the Telepass via the radio frequency unit and are transferred to a central computer via a network server in order to facilitate the preparation of a monthly debit statement that is provided to each user. The user reviews the statement to verify that such trips were made and possibly utilizes the statement for business record keeping and reimbursement.

The Autostrade began using the Telepass system in 1990 along a 750-kilometer corridor between Milan, Rome and Naples. On this corridor, Telepass toll gates stand at both entry and exit points because toll charges are based on distance as well as vehicle classification. Treadles count axles and infrared devices measure the vehicle's frontal height in order to classify a vehicle. Similar "read/write" systems are being examined for use in the metropolitan areas of Boston and New York City.

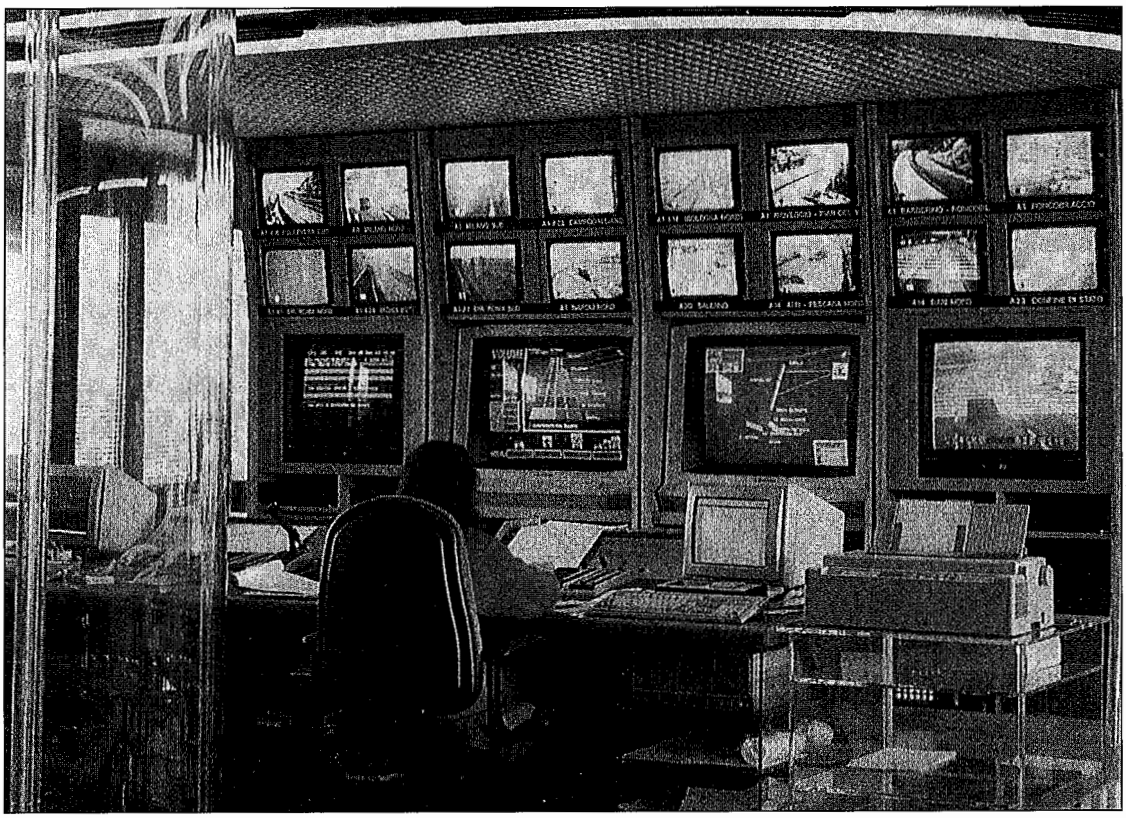


FIGURE 5. Autostrade staff in a traffic control center.

The Autostrade has used the Telepass since 1992 on about 60 kilometers of highway in the Milan-Laghi area where the distance is fixed between tolls and the fares vary only by vehicle classification. A toll gate is required only at a single point. A similar "read only," RF-based toll collection system has been employed by the Oklahoma and Texas turnpike authorities over the last several years.

At present, there are over 80,000 Telepass users and Autostrade officials expect this number to increase as the result of expanding Telepass service. Telepass users tend to be former current account ViaCard users. Autostrade pays about $40 for each two-way communication box and presently provides them to motorists free of charge.

Lessons Learned. As the use of electronic toll collection continues in Oklahoma and Texas, and as systems are developed in California, Florida, New York, Massachusetts and other states, interest will grow in this country in applying advanced technology to collect tolls. Perhaps some of the following lessons learned by the Autostrade will help federal and state highway officials:

Electronic toll collection can reduce delays at toll gates substantially. Autostrade officials estimate that typical transaction times are: 30 to 60 seconds for cash; 20 to 40 seconds for the ViaCard; and, two to three seconds for the Telepass. Time savings increase significantly when waiting times in queues during peak traffic periods are considered. (The time of a transaction includes the time the vehicle takes to travel through the toll gate.)

The fact that a stop toll system (ViaCard) was in place facilitated the implementation of a non-stop system (Telepass). Because the typical Telepass user is a former ViaCard user, issues of billing and accounting, privacy and customer acceptance already had been addressed.

Designing and implementing the ViaCard system addressed many of the issues, dilemmas

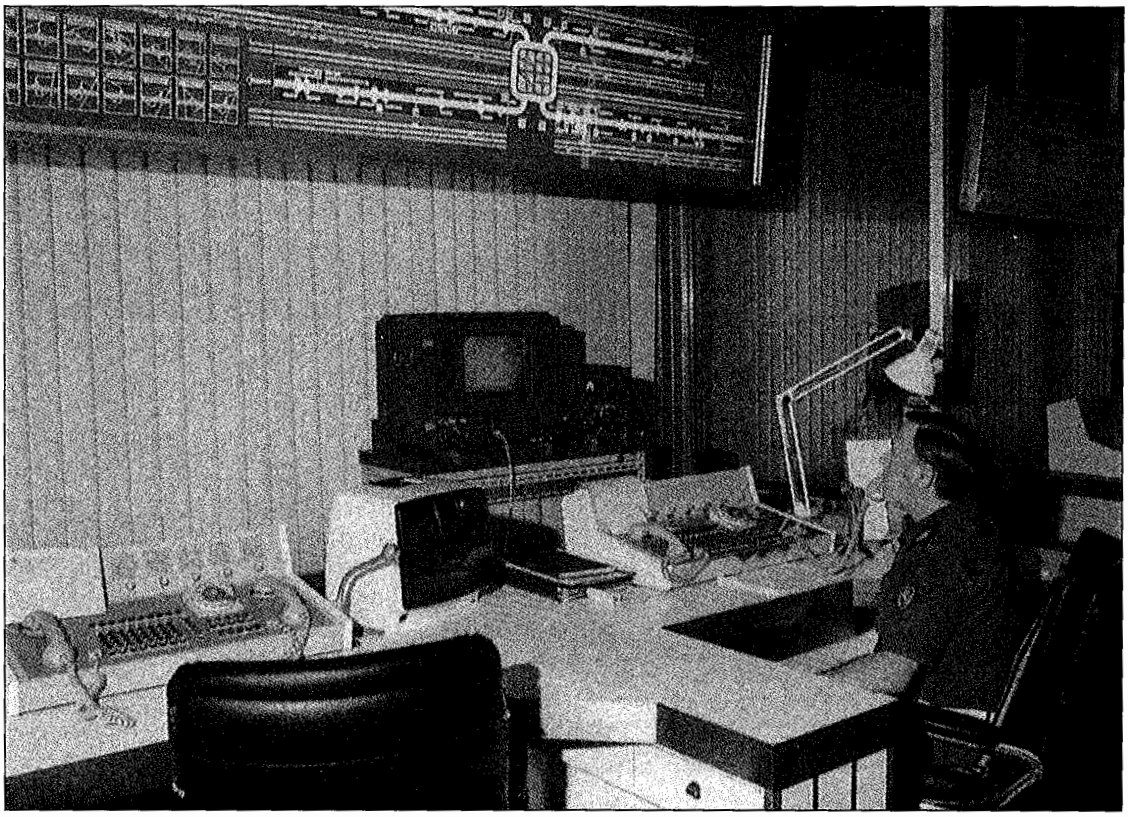


FIGURE 6. The police communicates directly with Autostrade staff at a traffic control center.

and institutional controversies regarding automatic vehicle identification. Consequently, when advanced technology became available to develop a non-stop toll collection system, the primary problem remaining was only one of technical communication between the vehicle and the central computer.

The cost of a Telepass lane, including pavement markings and equipment, is about $100,000; a ViaCard lane costs about $67,000.

In the early stages of ViaCard research and development, Autostrade officials worked with labor unions to ensure that toll collectors would remain productively employed. Some former toll collectors became ViaCard/Telepass sales personnel, while others were trained to assume comparable and, in some cases, higher positions.

Incident Management

Prior to commencing operations, Italian toll road agencies are required by law to establish a radio communication link with national police authorities and to create a written agreement for towing and related services with the ACI, Italy's national automobile association.

Because of the level and nature of traffic along Italy's more than 5,000-kilometer tollway system, responding to traffic incidents efficiently and with the utmost concern for safety has been considered essential by the Italian government. The toll roads in Italy at present accommodate some 50 billion vehicle kilometers per year, about 20 percent of which are for commercial vehicles. Despite a steady increase in usage, accident levels along the Autostrade have reached an all time low that, according to some officials, is attributed in part to a conscious effort to improve incident management activities on a continuing basis.

Fault-Tolerant Information System. An integral part of the Autostrade's current incident management program is the *sistema informativo* — its information system. This system consists

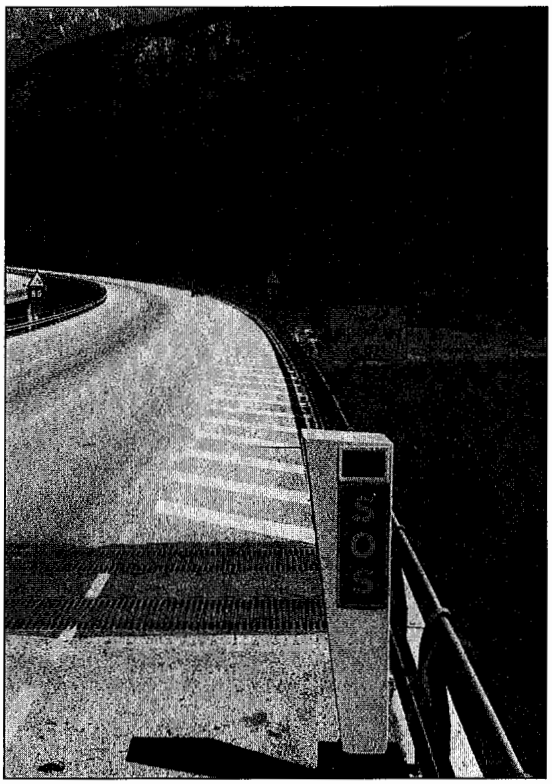


FIGURE 7. An SOS callbox.

of a central host computer to which are connected nine fault-tolerant mini-computers. These computers have secondary power sources in the event of a power outage. Each mini-computer is located in the nine Autostrade traffic control centers and is operated by Autostrade staff (see Figure 5). These control centers were established the late 1970s. Also located in each control center are police staff that communicate directly with the Autostrade staff and other police patrolling the Autostrade (see Figure 6).

Detection & Verification. A major function of each control center is to assist in identifying incidents and in determining the location and nature of each incident. With the use of closed-circuit television equipment and video cameras, the Autostrade staff has been monitoring traffic flows at key locations for the last five years. Selected video cameras can be controlled from the traffic control center to pan, zoom and focus on the incident location. Depending on the nature of the incident, the Autostrade staff in turn communicates the necessary information to the police or to a service vehicle.

Other sources employed by Autostrade staff to detect and verify incidents include information from motorists with in-vehicle telephone, roadway maintenance crews and SOS callboxes. These callboxes were introduced in the early 1970s and are located every 1.8 kilometers along the roadside (see Figure 7). Motorists using the callbox can request directly either a service vehicle or an emergency vehicle, including police, fire or ambulance services.

Traveler Information. Another major function of the control center is to provide motorists with information regarding traffic conditions ahead. Variable message signs controlled by Autostrade staff are located at major toll plazas and along the roadway (see Figure 8). Information is provided continuously alerting drivers to incidents ahead and the need to reduce speed due to perhaps an accident, maintenance work, inclement weather or poor visibility. Operating speeds are monitored and estimated by the control center with the use of special video cameras and image processing software.

In selected locations along the roadway where fog is a common problem, a fog detection system is employed to measure visibility. This information is also transmitted via computer to the control center and, if necessary, is displayed on variable message signs. In addition, the control center is provided weather information on a continuous basis via satellite and a remote weather control system with the cooperation of the Italian Air Force.

Individuals can also obtain traveler information from a local radio station (103.3 FM) or the national "Televideo" television channel. Interested individuals can also telephone the national traffic center in Rome or the local Autostrade office and obtain recorded information regarding traffic conditions. Motorists may also acquire traveler information at locations called *Punto Blu* (the Blue Point) — an information center located at each service area on the roadway (see Figure 9).

Training. In order to prepare Autostrade staff to handle the day-to-day duties and responsibilities in the traffic control centers, the

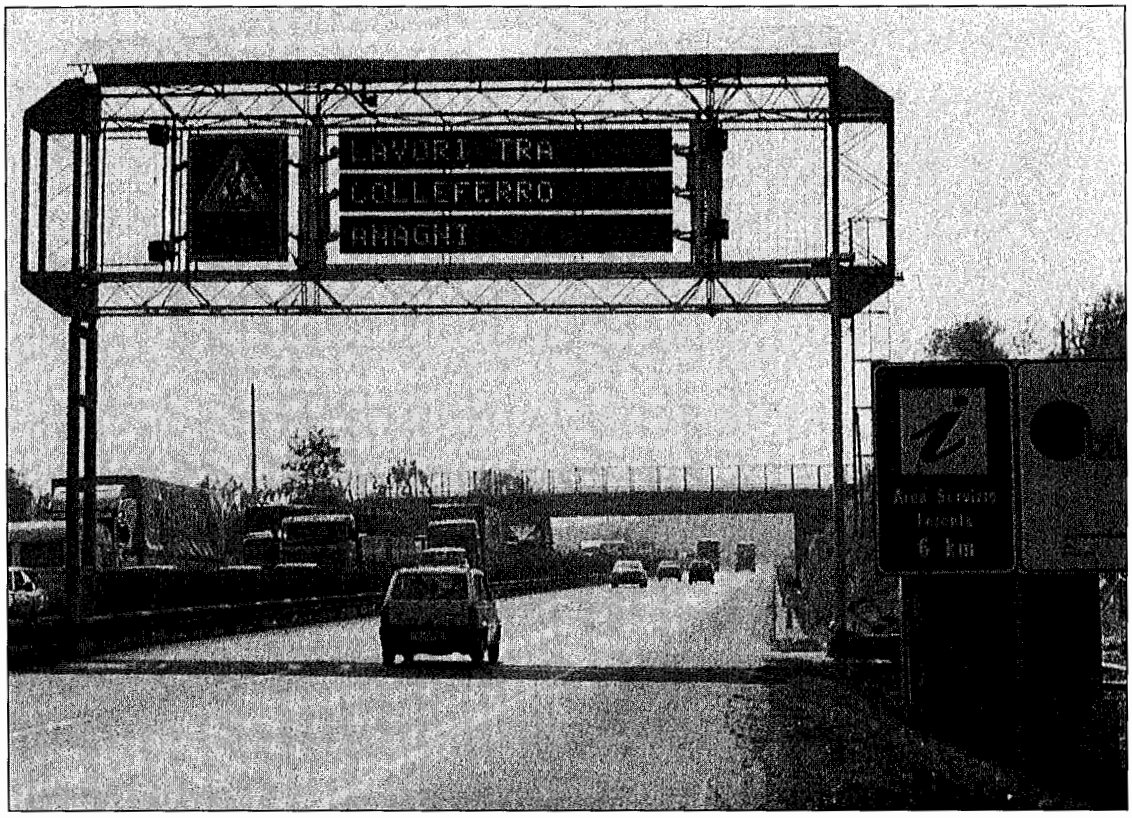


FIGURE 8. A variable message sign indicating that maintenance work is being carried out along a stretch of roadway.

Autostrade has established a training program in Rome. As part of this program, experienced Autostrade staff serve as instructors along with highway transportation expert consultants. Individuals enrolled in these training courses are required to work a designated number of hours in a control center under the supervision of a regular control staff member.

Summary, Conclusions & Future Directions

Autostrade officials expect significant increases in the use of ViaCard and Telepass over the next two years. In addition, officials plan to continue to enhance the incident management program with the use of further technological advancements.

Autostrade is collaborating with other transportation providers in Italy so that Telepass can be used on other toll roads in the country, as well as for air, rail, port and parking facilities. Autostrade officials also serve on several DRIVE projects. (DRIVE is the Intelligent Vehicle Highway Society research effort that is administered under the auspices of the European Commission.) Autostrade officials, together with toll road officials from other European countries, are exploring the possibility of establishing uniform functional and technical specifications for ETTM for all of Western Europe.

Autostrade staff are currently exploring the use of advanced computer-aided techniques to assist control center staff in making decisions in response to various types of incidents and accidents. Expert systems and other computerized methods and concepts are being considered.

Autostrade staff and Italian university researchers, in conjunction with engineers and computer scientists in Germany and France, are participating in the DRIVE-sponsored Gemini Project. This project's objective is to develop and test an integrated driver informa-

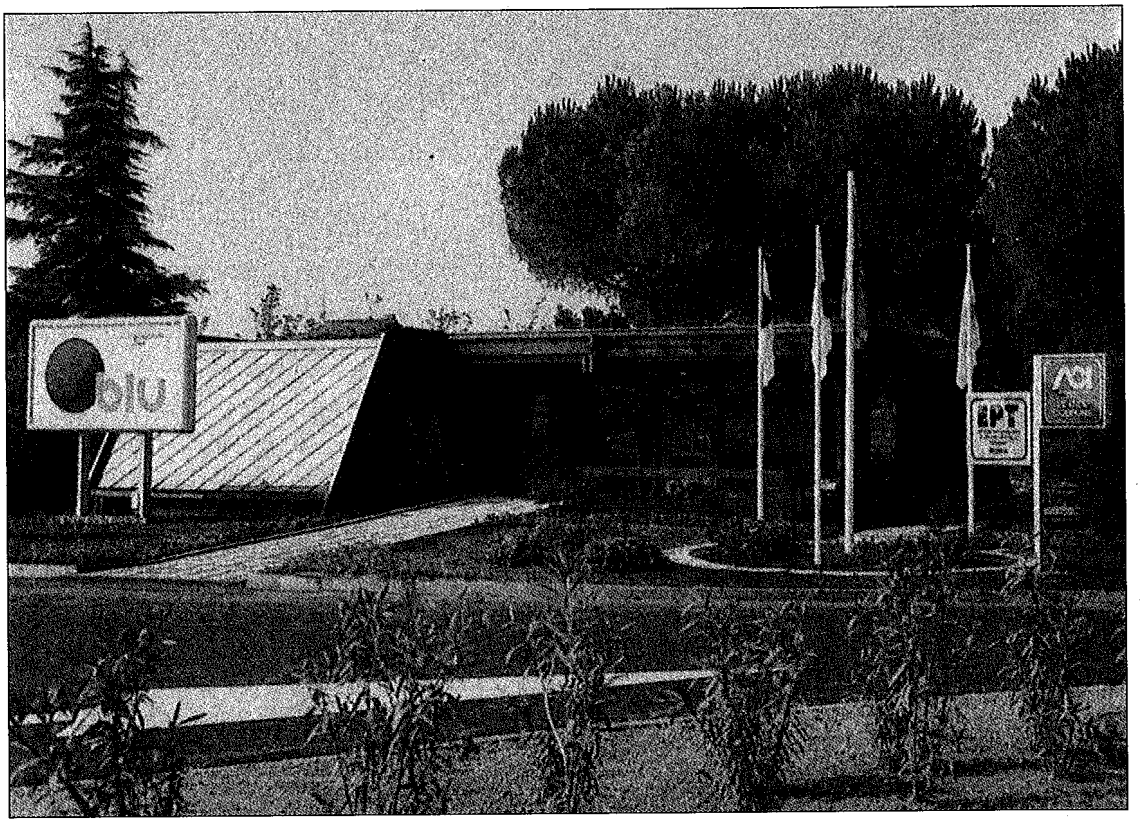


FIGURE 9. A *Punto Blu* information center.

tion system using radio data system traffic channel (RDS-TMC) and variable message sign (VMS) networks. Gemini's overall purpose is to contribute to the design of a pan-European driver information system that distributes non-contradictory and mutually confirming information that, in turn, leads to improvements in RDS-TMC and VMS standardization and protocols.

JOHN COLLURA *is a registered professional engineer in Massachusetts and a Professor of Civil Engineering at the University of Massachusetts in Amherst. He recently served as a Distinguished Faculty Fellow under the auspices of the Intelligent Vehicle Highway Society of America, and worked in conjunction with the Autostrade, Italy's largest toll road agency.*

The Use of Simulation Software for a Power Plant Construction Project

Computer simulation programs can be used in addition to historical methods of planning and field design for large-scale construction projects.

REED W. NIELSEN & ARTHUR K. STOVER

As three-dimensional (3-D) design becomes more and more common in the engineering/construction business, opportunities are increasing for its use in the field as a design/planning tool as well as a visualization tool. Two construction simulation software programs were used in the field design and the construction planning efforts of a fossil-fired power plant, thereby taking advantage of information available in the project's 3-D design model.

One of the principle purposes for the use of construction simulation software on this construction project was to study the effect of using computer-aided engineering tools at an actual construction site. Partial funding to offset additional costs for training and model preparation was provided through a contractual agreement with the Civil Engineering Research Foundation (CERF).

One of the two simulation programs used on the project is a 3-D, real-time simulation and animation software system. This real-time simulation system provides the user with the ability to move through a project as if it were a completed facility. This system allowed the construction field personnel to familiarize themselves with the physical configuration of the plant, and to use that familiarity to optimize field planning for equipment installation, work sequencing and field design.

Examples of the program's visual capabilities are provided in Figure 1, which shows a view across the turbine deck toward the boiler building and pipe chase, and Figure 2, which shows the relationship of boiler roof penetrations to structural steel, with notes identifying spool designators, lengths and weights.

The other program used in the field at the project is a 3-D computer-aided construction planning tool. This 3-D planning tool also allows the user to move through the design model, but adds the capabilities to develop schedules and to monitor work status/progress. Generated from this program, Figure 3 shows a screen image of the boiler and structural steel schedule simulation that was developed for the project at the site. The 3-D plan-

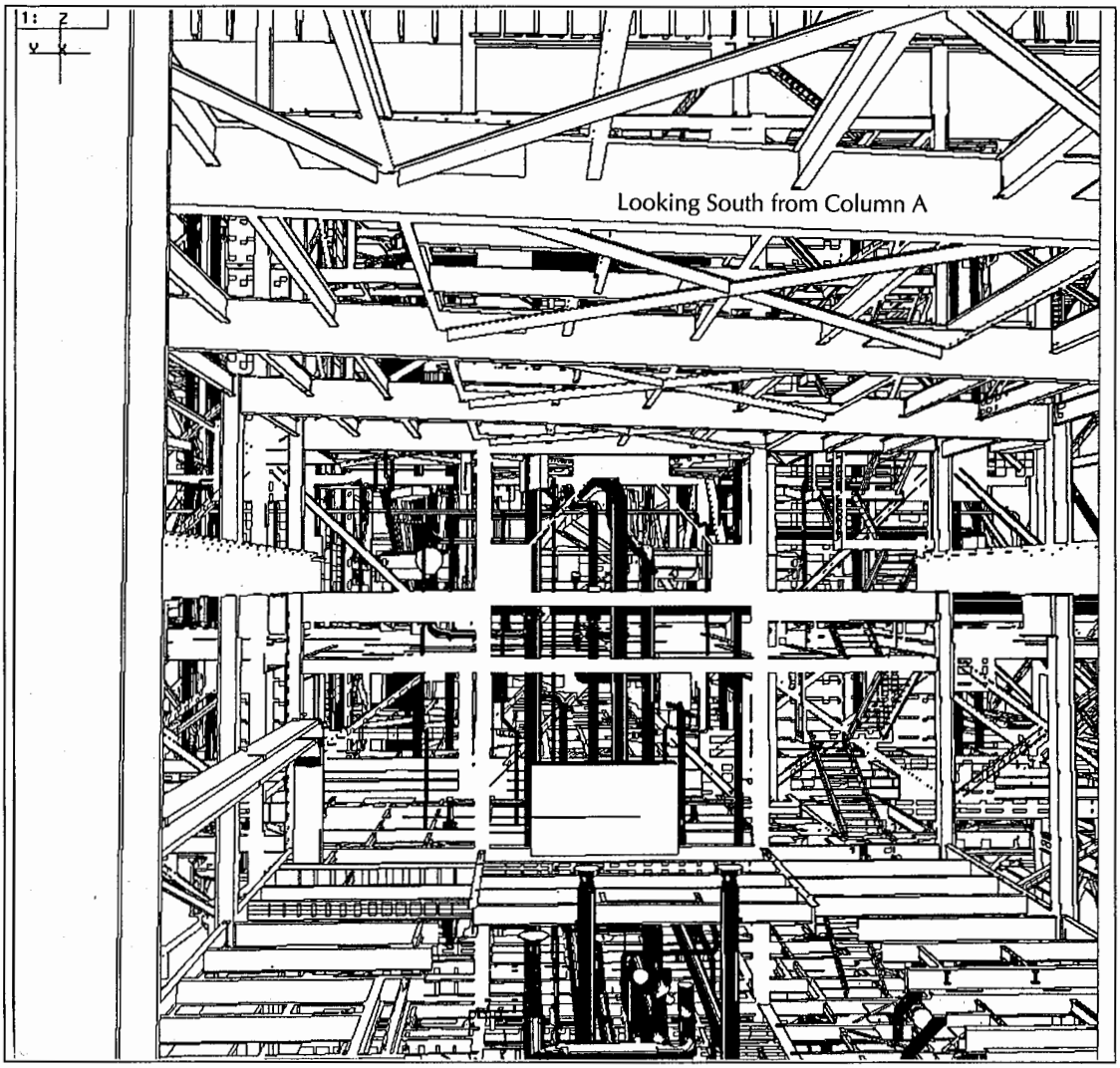


FIGURE 1. A view across the turbine deck toward the boiler building and pipe chase.

ning tool makes it possible to prepare step-by-step schedules on the computer screen using the design model. It is then possible to display animated work sequences based on the schedule, and to look for dynamic construction interferences that are caused by the schedule, since scheduled activities drive the display of the model on the computer screen.

Both the real-time simulation system and the 3-D planning tool were installed at the construction site on a high-end graphics workstation that was delivered in September 1991. They were used by field engineers and superintendents. Design was performed at a remote site using a 3-D design package. Model updates were sent to the field approximately once a month from October 1991 to December 1992.

Plant Description

The 90 megawatt (MW) coal-burning power plant is located in the Scrubgrass township in Venango County, Pennsylvania, just north of Interstate 80 and approximately 20 miles east of Interstate 79. It consists of two 380,000-pound-per-hour circulating fluidized bed boilers that provide steam to a single turbine/generator. Its coal handling system is capable of handling up to 240 tons of waste coal an hour.

Design began in January 1991, and first concrete was placed in May of the same year. Struc-

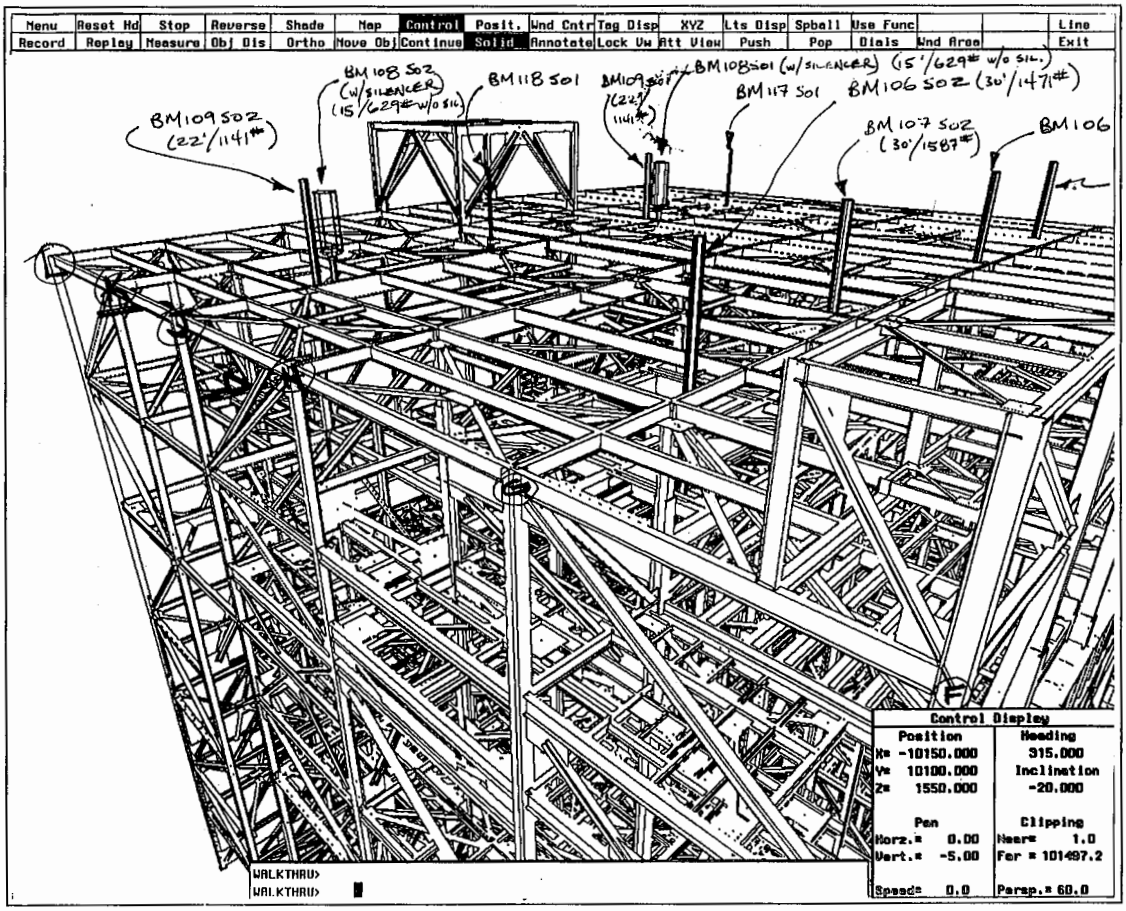


FIGURE 2. Boiler roof penetrations to structural steel (with field notations).

tural steel erection began in August 1991, and the steam drums were set in place in December 1991 and January 1992. Boiler hydro testing was completed in July 1992 and steam blows performed in December 1992. During peak construction there were over 500 craftsmen working at the site. The plant was declared in operation in June 1993.

The following quantities provide additional description of the size of the facility:

- Concrete: 11,000 cubic yards
- Large pipe (above ground): 15,000 feet
- Small pipe (above ground): 43,000 feet
- Conduit: 62,000 feet
- Wire and cable: 743,000 feet
- Terminations: 29,000 each

Training

Because the two simulation software packages would be used by field engineers and superintendents, some training was obviously necessary. In the end, training on the use of the 3-D planning tool was more formal. It was provided by an engineer who spent a total of about three weeks at the site. While on the site, the engineer arranged training to fit the schedules of potential users. In most cases, however, after a brief introduction, this training was limited to helping individuals use the computer in order to accomplish a desired work task.

The real-time simulation system, on the other hand, was for the most part self taught. One of the authors had previous limited exposure to the program that allowed him, with telephone instructions from off-site personnel that were familiar with the system, to start the computer and display the design model on the screen. A copy of the real-time simulation system instructional manual was shipped to the

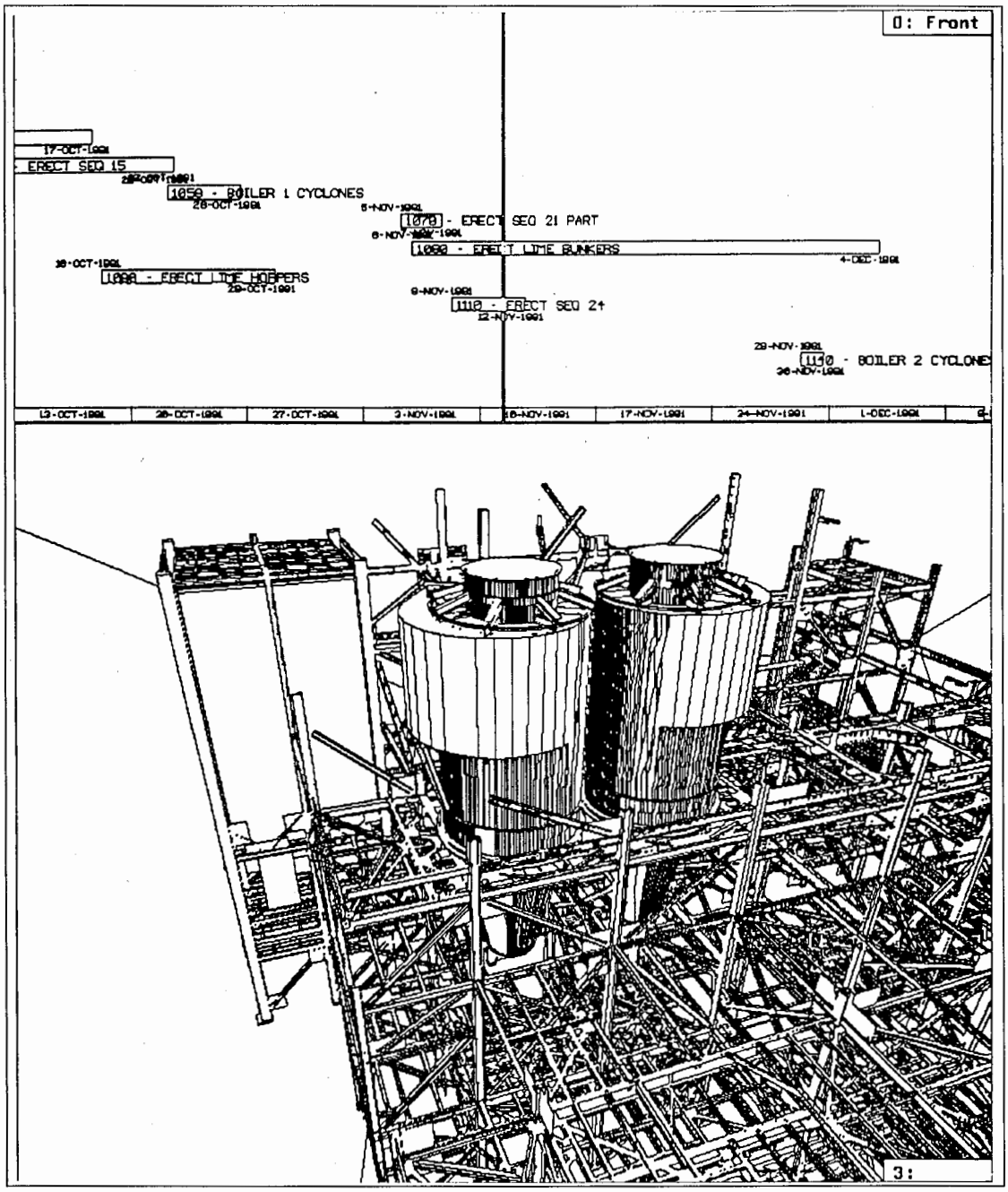


FIGURE 3. Screen image showing the boiler and structural steel schedule simulation.

site and, with additional telephone instruction, field personnel were able to successfully use most of the functional parts of the program in order to complete the desired tasks.

Simulation Uses

The simulation software was used:

- to help in the erection planning of pipe commodities and electrical cable tray;
- in the field design of plant lighting; and,
- for after-the-fact confirmation of the sequencing previously decided upon for structural steel and boiler erection.

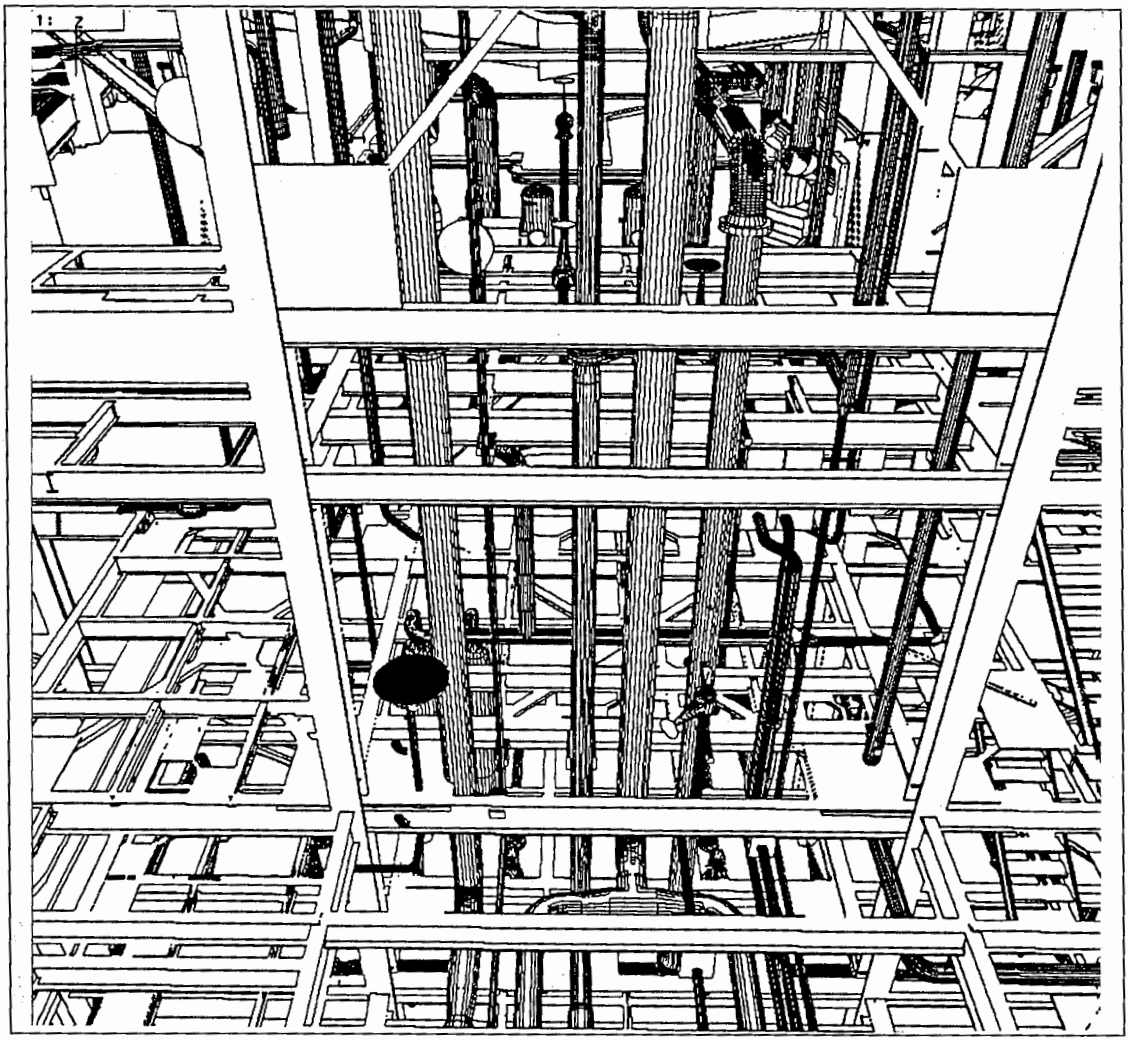


FIGURE 4. A view of the pipe chase between the turbine and boiler buildings.

Piping. The majority of carbon and alloy pipe was designed and fabricated off site. As small pipe isometrics were released from engineering, all small pipe spooling was established by field engineers who were using the design model to determine field weld location and to maximize spool length and configuration with respect to the other commodities shown on the model. Critical large-bore pipe spools were also checked for field weld locations to confirm the planned erection methods. This effort minimized the amount of field modification required to transport the spools to their point of installation. It also ensured that the maximum amount of prefabrication was being performed.

Overly congested piping areas were viewed to determine delivery and erection sequencing and to maximize scaffold usage. Figure 4 shows a view of the pipe chase located between the turbine and boiler buildings. Some interferences were discovered during the course of this activity, which permitted rework of the associated piping at the shop prior to delivery. Figure 5 illustrates this pipe to pipe interference.

Where isometric drawings did not detail specific valve operator and handwheel locations the model was consulted in order to determine the most appropriate configuration given the surrounding equipment, structural steel configuration and access.

In many cases, when the piping was scheduled to be installed, "screen dumps" (print-

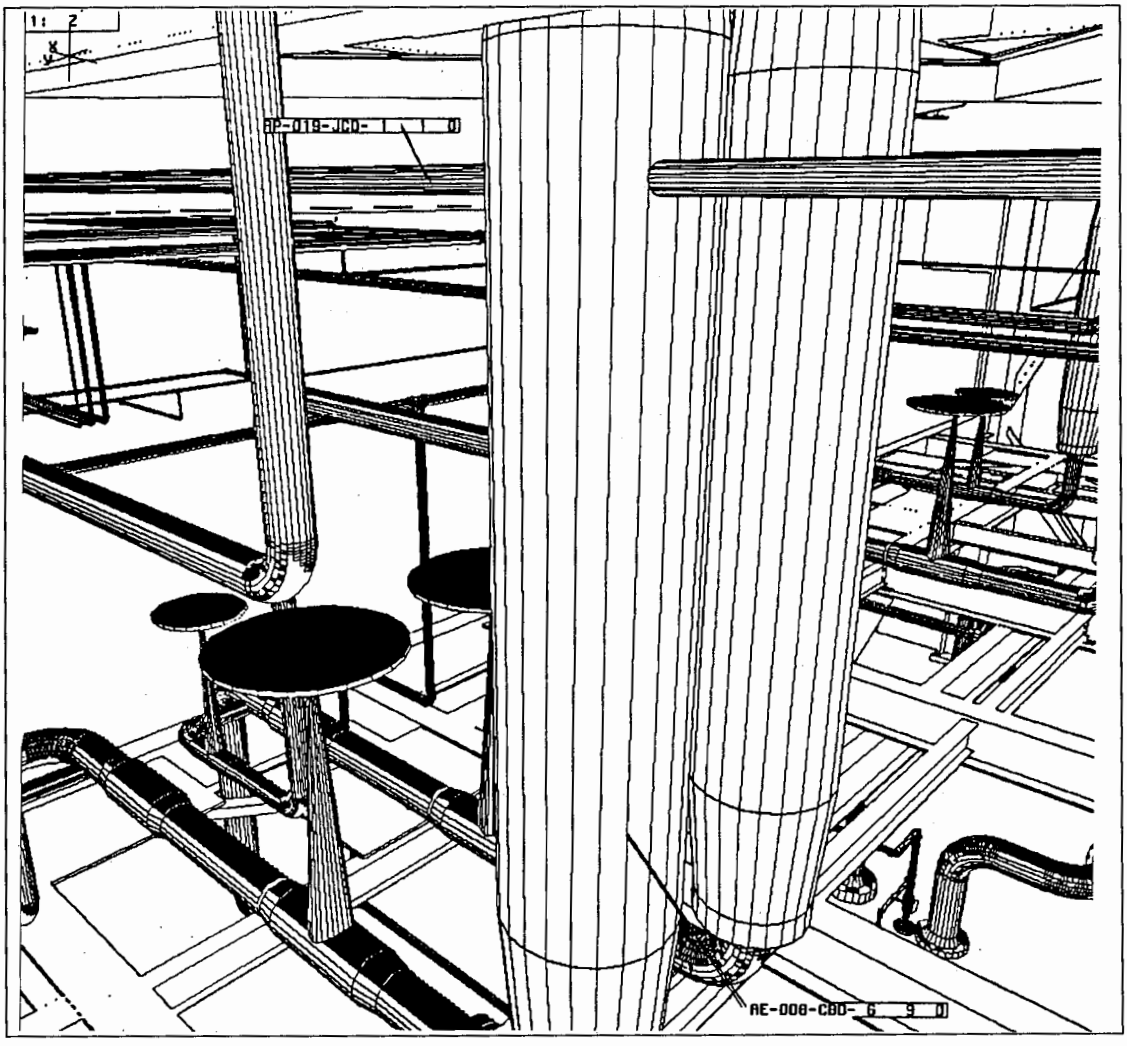


FIGURE 5. Pipe to pipe interference.

outs of the screen image) of the particular spools and areas from the model were made. Figure 6 is an example of one of these images. It shows pipe located near the coal bunkers. Visual aids such as these provided considerable benefit to the pipe fitter who was installing the pipe. These screen dumps could be found throughout the facility taped to office walls, work supervisor shacks and structural columns. On several occasions, requests came in from the field supervisors and personnel for "pictures" of a specific area or system.

Electrical. The model was used to field design lighting in the boiler and turbine areas. The engineer performing this design spent many hours at night after work reviewing the areas and elevations not only to place the lights in locations of most benefit, but also to assure that once they were installed they would not need to be moved as other components were installed. Only one light fixture from over 750 required relocation during the course of construction.

Screen dumps that showed the cable tray locations with respect to structural steel were also provided to electricians. The information contained in these images helped in the location of cable tray hangers as well as helping field personnel better interpret the tray drawings.

Because of the availability of the electronic model, the project was able to start cable tray installation prior to the installation of above ground piping. This procedure is not common

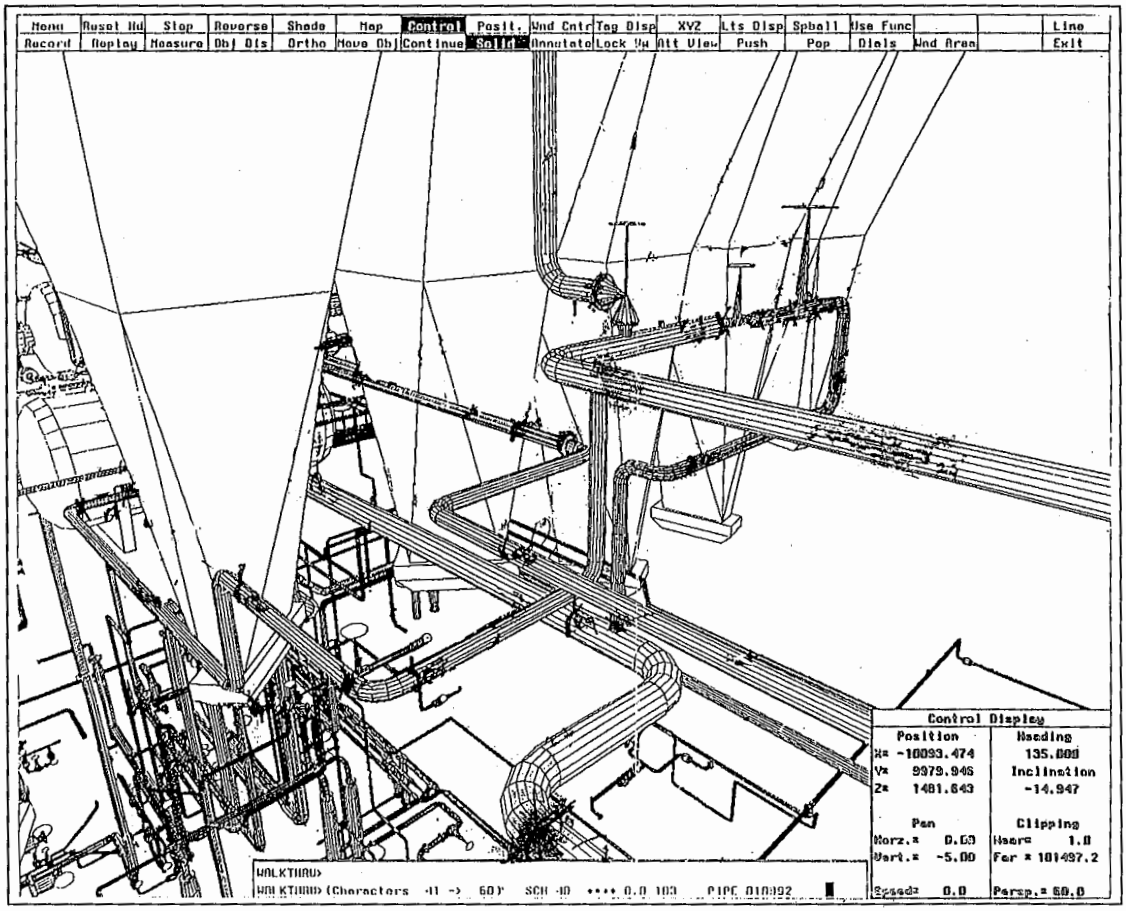


FIGURE 6. A screen dump showing pipe located near the coal bunkers.

and was successful because of the ability to use and manipulate the 3-D image. Historically, when this sequencing is attempted in a vertical plant of this nature with multiple commodities, construction interferences (that occur when moving equipment into an area prior to installation) are common and often expensive.

Reviewing the model prior to issuing work to the field permitted early identification and resolution of a number of design and field interferences, thus avoiding costly rework later on.

Structural Steel & Boiler Component Erection. Because the decision to use simulation software was not made until early summer of 1991, and the hardware not available on site until September 1991, most of the erection planning for structural steel and boiler components had already been finalized. On a fast track project with a short design and construction schedule, boiler component delivery schedules must be made shortly after the start of design. Structural steel schedules must also be established soon after structural steel design has begun and before a detail model is available.

To show the capabilities of the 3-D planning tool, the schedule was replicated in the software and a video was produced to portray what had already been developed. It helped visualize the sequence of boiler component and structural steel erection and clearly validated the established structural steel "hold out" areas for other plant equipment installation. This schedule was put together on the site by a knowledgeable simulation engineer with input from field engineers and superintendents. It showed capabilities of the software that had not been fully explored in other field efforts.

Lessons Learned

The fast track nature of jobs such as the Scrub-

grass project require that the use of simulation software be thoroughly planned and integrated into the design, procurement and construction schedule. It must be started at the beginning to be fully successful. Late implementation — though beneficial in field layout, planning and design — does not allow full use of the software in high-level planning such as major component (*e.g.*, boiler component, structural steel, etc.) deliveries.

The simulation package must be relatively simple to use. The 3-D planning tool was not used nearly as extensively as the real-time simulation system due to its complexity and, therefore, its difficulty to learn and use. As a result, much of its schedule development and review functionality did not provide significant benefits to the project. However, incorporating lessons learned at Scrubgrass, a recent release of the 3-D planning tool makes it easier to use.

The real-time simulation system, because it was relatively simple to use, did not require much in-depth training. As a result, the 3-D model was accessed much more frequently with that software. Ease of use was traded off for schedule development and playback functionality.

Future field applications should have the capability to print larger than 8.5- by 11-inch images. At a minimum, the capability to create B-size drawings should be available for planning purposes and field distribution.

A full-time custodian should be considered for future jobs who is fully trained in all of the software's features. A resident "expert" will allow projects to obtain maximum benefit from the powerful features that are available in today's computer simulation software.

Conclusion

There is a place for computer simulation in the field. The ability to use the 3-D design model gives field personnel added information that can be used to enhance field planning and erection methods. This model, used with simulation software, provides field personnel with the ability to selectively overlay commodities and components for in-process and final location interference checks as well as for area familiarization and visualization purposes. This feature was particularly important at the Scrubgrass project given the large volume of field-designed and routed installations and has the potential to be of benefit on most projects today.

ACKNOWLEDGMENTS — *Detailed documentation of the project is contained in CERF report 92-N6001, "Computer Aided Engineering (CAE): A Promising Tool for Improving Construction Site Productivity," October 1992. The real-time simulation system used on the project was Walkthru and the 3-D planning tool was Construction CAE. Bechtel developed both simulation packages as well as the 3-D design software used on the project. The simulation software was installed and operated on a Silicon Graphics Iris 4-D workstation. The power plant was designed by the San Francisco office of Bechtel Power Corp. and constructed, using union craftsmen, by Bechtel Construction Co., also based in San Francisco. The owner is U. S. Generating Co. located in Bethesda, Maryland. The fluidized bed boilers were manufactured by Tampella Power. The turbine/generator was manufactured by GEC/Alsthom. Training in Construction CAE was provided by Eric Jost, an engineer from Bechtel's Los Angeles regional office. Field notes were maintained by the following: Art Stover, Scrubgrass Project Field Engineer; Bill Leeland, Scrubgrass Piping Superintendent; and, Bryan Kerr, Scrubgrass Lead Field Electrical Engineer.*

REED W. NIELSEN *was the Bechtel Site Manager at the Scrubgrass power plant. Prior to that assignment, he was Manager of Construction Technologies for Bechtel Group, Inc., in San Francisco. Earlier assignments included contract, craft engineering and field engineering roles at fossil and nuclear plant construction. He graduated from Arizona State University with a B.S. in construction, and is a registered civil engineer in California and Utah.*

ARTHUR K. STOVER, *Project Field Engineer for Bechtel Construction, was responsible for planning, organizing and controlling the activities of the Field Engineering organization during the construction of the Scrubgrass power plant. He has worked on numerous power plant construction projects. He graduated from Montana State University with a degree in construction engineering, and has spent the last 14 years with Bechtel.*

Reducing Seismic Risk in Massachusetts

A program that evaluates the seismic risk of existing buildings is instrumental in developing structural modifications that will reduce seismic risk.

STEVEN P. MCELLIGOTT, JAMES R. GAGNON & CHRISTOPHER H. CONLEY

No single factor clearly defines seismic risk in Massachusetts, nor for that matter for the rest of the Northeast United States. It is only when several factors are considered together that the risk becomes clear. This state of affairs presents a clear problem — the general public in the region is not aware of all these factors and, therefore, does not perceive the very real threat from earthquakes.

Viewed together, the following four factors portend a potential disaster. The first factor is that the mechanics of earthquakes in the Northeast are not well understood. There does not appear to be a correlation between epicenters and geologic features such as faults. This lack of correlation implies that an earthquake may happen anywhere in the region, and at any time. There is also no limit on the magnitude of the earthquake that may occur. Even though there have not been any significant earthquakes affecting the region in the recent past, history shows that significant earthquakes have occurred. In the colonial period the region was very active. Many earthquakes were reported in the written records of the period. For instance, the following was written in reference to an earthquake in early June 1638:[1]

> "Between three and four in the afternoon, being clear, warm weather, the wind westerly, there was a great earthquake. It came with a noise like a continued thunder, or the rattling of coaches in London, but was presently gone. It was at Connecticut, at Naragansett, at Pascataquack, and all the parts round about. It shook the ships, which rode in the harbour, and all the islands, &c. The noise and the shakings continued about four minutes. The earth was unquiet twenty days after, by times."

The second factor, also associated with geology, is the high transmissibility of earthquake energy in the region, which translates into the potential for widespread damage. This transmissibility is hinted at in the above quotation, and is further illustrated by the reports that both the New Madrid, Missouri (1811-1812), and the Charleston, South Carolina (1886), earthquakes were felt in Boston.

The third factor is the general lack of seismic resistance in the building stock. While recent versions of the Massachusetts building code

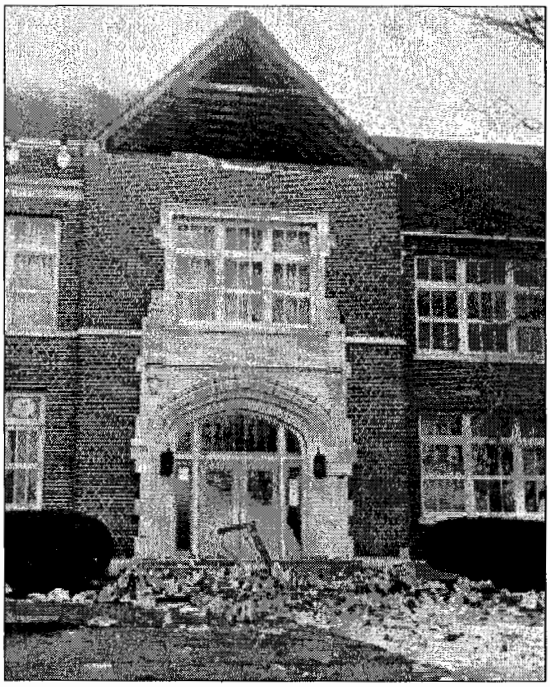


FIGURE 1. Destruction of unreinforced nonbearing structural elements from an earthquake.

have included seismic provisions, they pertain mainly to new construction. The majority of buildings in the state were not built with lateral loading in mind. Without regular seismic activity as a reminder (as occurs in California), the tendency is to ignore potential earthquake loading. The large number of unreinforced masonry structures heightens the threat to public safety. Under such conditions, even relatively mild earthquakes can result in significant risk for loss of life and property, as demonstrated by the March 1993 earthquake in Scotts Mills, Oregon.[2] Figure 1 shows damage that occurred to a high school from that magnitude 5.6 event. Imagine the consequences had a group of students been on the doorstep of the school.

The final factor is high population density. Maps of population in the U.S. clearly show population concentrations on the two coasts, with an especially dense band along the northeast coast. Were a large earthquake to occur (and there is no reason to believe it will not), the potential for a disaster is high due to the large number of structures that are vulnerable to earthquakes and the large number of people using those buildings.

Given the general lack of acceptance of seismic risk in the Northeast, it seems unlikely the public in the region will spend additional monies on seismic hazard reduction. Funds could be allocated from the current tax base, but given these economic times, that would probably mean cutting some other essential programs. Ultimately, the public will have to commit funds if large-scale remediation is to take place. However, such an allocation cannot occur until the public is sold on its necessity. Until that time, and as part of that selling effort, certain things can be done that can serve as the basis for large-scale remediation. These things can also help sell the public on the more cost-effective idea of taking advantage of opportunities for seismic retrofitting of essential buildings during other renovation activity.

The Beginnings of a Solution

In summary, a very focused effort at reducing seismic risk in Massachusetts involves a number of activities:

1. Conduct a rapid visual screening of essential facilities. This screening will help define the magnitude of the task, and will serve as a planning tool for implementing the rest of the program.
2. Perform a detailed seismic evaluation of typical structures from each group of essential facilities with the goal of identifying common features of the buildings in each group that make them seismically vulnerable. For example, the large openings found in the first story of every fire station make these buildings especially vulnerable.
3. Design retrofit techniques that are catered to the region and to specific types of buildings. It is hoped that techniques can be developed that will be usable, with minor variations, over and over again. At this point, a good estimate of retrofitting costs can be made.
4. Institute a widespread retrofit program for all essential facilities in the state. Much of this retrofitting can be done in conjunction with other programs. For instance, many of these essential facilities are also historically significant and are often restored/renovated as part of historic preser-

vation programs. Seismic retrofit should be incorporated into such programs. In many cases, it would only add moderately to the cost. Also, schools are often renovated and should at the same time be seismically retrofitted.

While implementing such programs, close attention should be paid to the actions of the federal government. The Earthquake Hazards Reduction Act of 1977 and Executive Order 12699 outline the actions the federal government is to take with regard to the buildings it owns and leases. Executive Order 12699 mandates that all new buildings must be designed and built in accordance with "appropriate seismic design and construction standards," and work is underway to implement this order. More significant in regard to scope and cost is the provision in Sec. 8. (a) in the Earthquake Hazards Reduction Act of 1977 which states that:

> "The President shall adopt, not later than December 1, 1994, standards for assessing and enhancing the seismic safety of existing buildings constructed for or leased by the Federal Government..."

Some valuable lessons will surely be learned by studying the implementation of this legislation. It should also be noted, as it was in the Earthquake Hazards Reduction Act of 1977, that much of the work done towards reducing seismic risk would also reduce the risk from other natural hazards such as hurricanes.

Work based on the first three steps noted above has begun on a smale scale in Massachusetts. The rapid screening of fire stations in Middlesex County described in the next section was conducted in the summer of 1992. This rapid screening was followed by a detailed seismic evaluation of one of the fire stations, and research on externally bonded reinforcement as a means to strengthen unreinforced masonry walls — also described below.

Rapid Screening of Fire Stations in Middlesex County

As previously noted, despite a significant earthquake hazard in the eastern United States, few structures in this region have been constructed to withstand the lateral loads that an earthquake may impose. The Earthquake Hazards Reduction Act of 1977 (amended November 1990) outlines several steps that can mitigate this threat, one of which is to "conduct seismic safety inspections of critical structures" that may fail in an earthquake.[3]

One tool designed for seismic safety inspections is the Federal Emergency Management Agency (FEMA) publication "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook" (FEMA-154), hereinafter noted as the RSP.[4] This document outlines a procedure to identify buildings that may sustain serious damage in an earthquake, and any that fail the screening are noted as requiring inspection by a professional engineer experienced in earthquake engineering.

The RSP document is intended for "local building officials, professional engineers, registered architects, building owners, emergency managers, and interested citizens."[4] Obviously, the range of expertise among this group could vary from none to complete. For this project, civil engineering undergraduate students were employed to exercise the RSP. Engineering students at the upper undergraduate level have been taught dynamics, basic structural analysis, and the characteristics of building materials — all of which make understanding and using the RSP easier.

The actual implementation of the RSP is simple since it is intended to be a "sidewalk survey" procedure based almost entirely on information gathered visually. Figure 2 shows the form that is filled out during an inspection. A building is identified based on its type of construction (unreinforced masonry, timber, steel, reinforced concrete, etc.) and is accordingly assigned a base seismic score. A seismically good construction type, such as wood, will have a high score, and a potentially poor type, such as unreinforced masonry, will have a low score. The basic scores can then be modified based on the physical characteristics of the building. For example, a building with a long wing projection may have its basic score reduced one point for a plan irregularity, since significant damage may occur at the re-entrant corner. A building may have its score increased only if it was built after seismic provisions were

ATC-21/ (NEHRP Map Areas 3,4, Moderate)
Rapid Visual Screening of Seismically Hazardous Buildings

Address_____
Zip_____
Other Identifiers_____
No. Stories_____ Year Built_____
Inspector_____ Date_____
Total Floor Area (sq. ft.)_____ Use_____
Building Name_____

OCCUPANCY

Residential	No. of Persons
Commercial	
Office	
Industrial	0-10
Pub. Assem.	11-100
School	100+
Govt. Bldg.	
Emer. Serv.	
Hist. Bldg.	

Non Structural Falling Hazard ☐

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data
DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RCSW)	C1 (MRF)	C2 (SW)	C3/S5 (URMINF)	PC1 (TU)	PC2	RM	URM
BASIC SCORE	6.0	4.0	3.0	6.0	4.0	3.0	3.5	2.0	3.5	2.0	3.5	2.0
HIGH RISE	N/A	-1.0	-0.5	N/A	-1.0	-0.5	-1.0	-1.0	N/A	0.0	-0.5	-0.5
POOR COND	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
VERT. IRREG.	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-0.5	-1.0	-1.0	-1.0	-0.5	-1.0
SOFT STORY	-1.0	-2.0	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-1.0	-2.0	-1.0
TORSION	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
PLAN IRREG.	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
POUNDING	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
LG.HVY.CLAD.	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
SHORT COL	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
PST BNMK YR	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 8-20 FLRS	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8
FINAL SCORE												

COMMENTS

Detailed Evaluation Required?

YES NO

FIGURE 2. A blank RSP form.

introduced in the governing building code — hence, determining the year of construction is vital. A final score is determined and compared to a cut-off score of 2.0. A score of 2.0 or less means that the building may be seismically hazardous and that it requires further evaluation.

In addition to the basic structural scores and modifiers, the RSP facilitates consideration of other characteristics of the building. The occupancy of the building and its general use can be noted in order to prioritize any further evaluations. A non-structural falling hazard can be

noted, and is typically a portion of the building (such as ornamentation like parapets and chimneys) that does not carry a load. Since these features are non-structural, they are often not anchored sufficiently to resist lateral loads and can be thrown from the building.

Teams of two or three have been most effective in implementing the RSP. The work of interviewing the building's occupants, photographing and measuring the building, and collecting global positioning data can be split among the team members.

Due to the large number of buildings in Massachusetts, some limits had to be placed on this project. The Earthquake Hazards Reduction Act of 1977 notes that priority for inspection should be given to "power generating plants, dams, hospitals, schools, public utilities, and other lifelines, public safety structures, high occupancy buildings, and other structures which are especially needed in time of disaster."[3] Similarly, the seismic provisions of the Uniform Building Code (UBC 1988) specifically mention the need for fire stations and police stations to withstand a higher level of lateral loading.[5]

By the above definitions, the number of essential buildings that should be screened is still in the thousands. To contain project duration to one summer, the decision was made to consider only fire stations. Fire stations are clearly essential facilities since fires often occur as a consequence of a seismic event, as witnessed by the aftermath of the 1906 San Francisco and 1989 Loma Prieta earthquakes.

Initially, the intention was to inspect all the fire stations in Massachusetts. Phone calls to the fire departments of each town in Middlesex County yielded an estimate of approximately 148 fire stations, though ultimately 151 were found. It was then decided to limit the screenings to fire stations in Middlesex County, which proved to contain a good cross section of urban, suburban and rural communities that have fire stations of various ages and types of construction.

As the inspections proceeded, the data that were collected in the field were stored on a personal computer. A word processor was used to reconstruct the RSP form, and the field data for each fire station were entered onto the computerized form. Highlighting (redlining) in the word processor mimicked marks made on the form in the field, and comments could be typed in the provided spaces.

The RSP calls for the inclusion of a photograph of the inspected building and a sketch of the building in plan. Room for two photographs was provided on the word-processed version of the form. The photographs were taken with a digital camera, each showing two of the four sides of the building whenever possible. Once the photographs were downloaded from the camera, they were touched up (sharpened, brightened, etc.) and converted into a format that could be imported into the word-processed form.

The plan view of the building was drawn with a CAD program using the measurements made in the field. These drawings were also converted into a format that could be imported into the form. The form is only one page in length, and has an average size of 65 kilobytes. Figure 3 is an example of a completed computerized form.

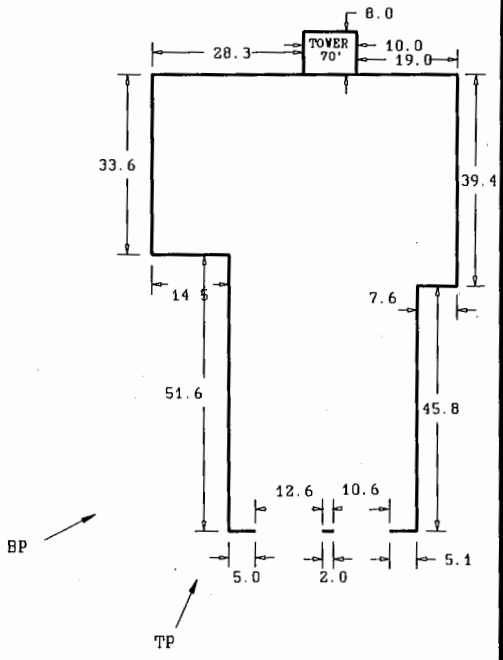
For the purposes of quick retrieval, the data from each form were stored in a database. Each fire station has its own record. Use of this database allowed queries to be made. It is a simple matter to use a logical query in order to get a listing of all fire stations built of unreinforced masonry, all the fire stations built before 1975 (the year that earthquake provisions were included in the Massachusetts building code), all fire stations meeting both of these criteria, or any other query desired for which data have been collected.

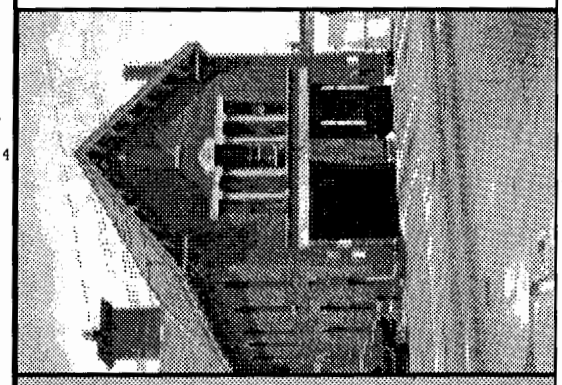
A geographic information system (GIS) was also used to present the results of the inspections. A GIS is a spatial database that links information about a feature on the earth's surface to the location of that feature. As noted previously, a global positioning unit was used to collect data at each fire station which yielded a longitude and latitude of each fire station that, with differential corrections, are within ten to 30 feet of the fire station's true location. The GIS converted these locations into a point coverage (a digital map composed of point locations) and linked the location of each fire station to its record in the database. The GIS can display the relative position of each fire station and, by selecting a fire station's icon, the record for that fire station can be displayed showing the RSP data.

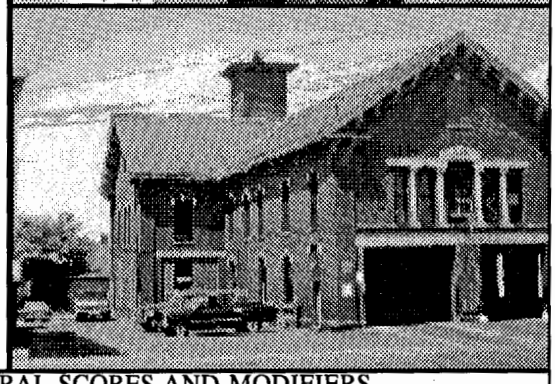
In order to further clarify the location of a fire

ATC-21/ (NEHRP Map Areas 3,4, Moderate)
Rapid Visual Screening of Seismically Hazardous Buildings

Address Branch Street LOWELL
Zip 01851
Other Identifiers Lowell Fire Dept., Station #2
No. Stories 2 + tower Year Built 1877
Inspector S. McElligott Date 4-10-92
Total Floor Area (sq. ft.) 7600 * Use FS
Building Name Station #2







OCCUPANCY

Residential	No. of
Commercial	Persons
Office	
Industrial	0-10
Pub. Assem.	11-100
School	100+
Govt. Bldg.	
Emer. Serv.	
Hist. Bldg.	

Non Structural Falling Hazard ☑

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data
DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RCSW)	C1 (MRF)	C2 (SW)	C3/S5 (URMINF)	PC1 (TU)	PC2	RM	URM
BASIC SCORE	6.0	4.0	3.0	6.0	4.0	3.0	3.5	2.0	3.5	2.0	3.5	2.0
HIGH RISE	N/A	-1.0	-0.5	N/A	-1.0	-0.5	-1.0	-1.0	N/A	0.0	-0.5	-0.5
POOR COND	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
VERT. IRREG.	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-0.5	-1.0	-1.0	-1.0	-0.5	-1.0
SOFT STORY	-1.0	-2.0	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-1.0	-2.0	-1.0
TORSION	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
PLAN IRREG.	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
POUNDING	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
LG.HVY.CLAD.	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
SHORT COL	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
PST BNMK YR	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 8-20 FLRS	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8
FINAL SCORE												-0.5

COMMENTS
-Unreinforced Chimney on right side of building
-2 different constructions (front and back) both are URM
-Floor to wall anchorage is provided, however not sufficient to protect from seismic loading (too small + sparse)

Detailed Evaluation Required? YES NO

FIGURE 3. A completed RSP form (for the Branch Street Fire Station).

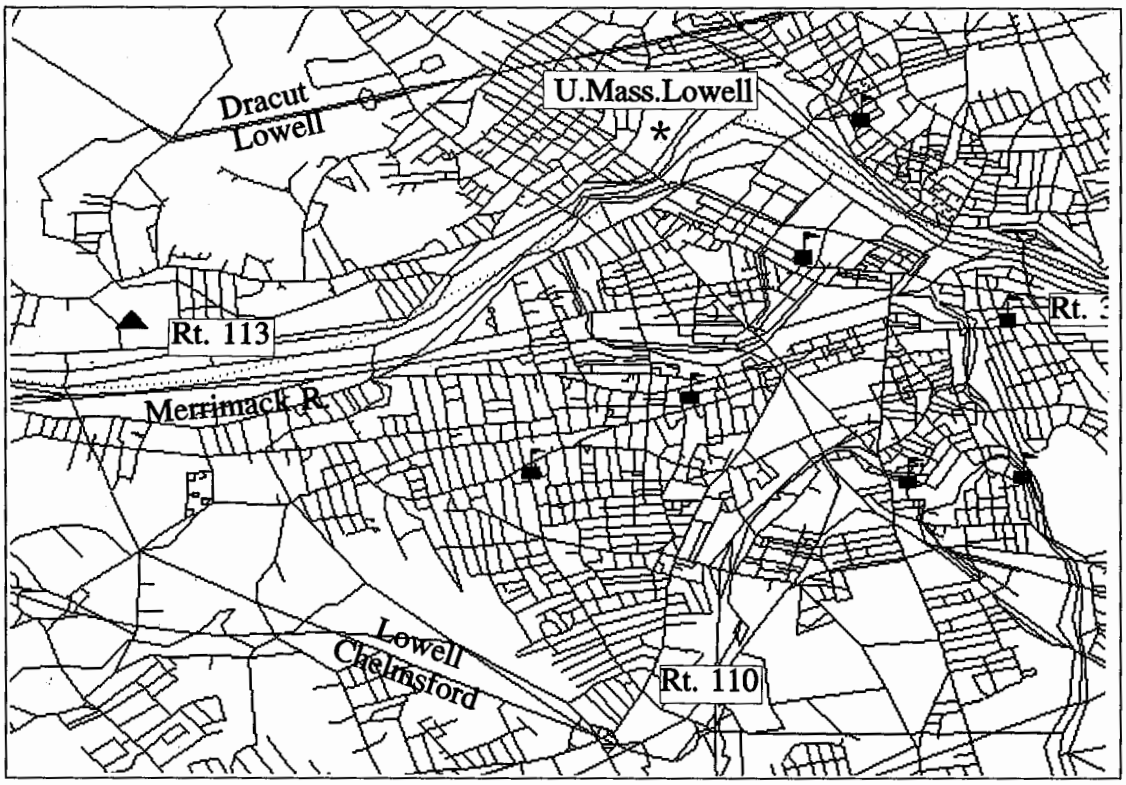


FIGURE 4. TIGER/Line digital map file for Lowell.

station to a user unfamiliar with the GIS database, a digitized street map of Middlesex county is also used. Data for the map were obtained from TIGER/Line data files from the U.S. Census Bureau that are stored on CD-ROM. The GIS converted the data file into a line coverage (a digital map composed of lines). The coverage for the fire stations could then be laid on top of the coverage for the street map, and specific data for either the fire stations or the streets can be displayed. Since the GIS program is large, it does not execute quickly on a personal computer. Therefore, for the purposes of just displaying the coverages and the databases, a quicker program with read-only capabilities is used.

The coverage of streets in the county is displayed and then the coverage of the fire stations is displayed on top of the streets. Figures 4, 5 and 6 demonstrate the use of the spatial database. The user can zoom into a particular city, such as Lowell, as shown in Figure 4. The text labels were not originally displayed and were added to clarify the street map. There are eight fire stations in Lowell, seven of which failed the RSP inspection and are represented by the flagged boxes. The one fire station that passed is represented by a solid triangle. A user can select a fire station with a mouse and a portion of the fire station's database record is displayed as shown in Figure 5. The database display can be blown up to full screen size as shown in Figure 6, and all the data from the RSP form, except for the photographs and plan drawing, can be read.

The data from the inspections yielded some interesting results. Of the total of 151 fire stations inspected, 130 received scores of 2.0 or less, indicating that the structure may present a threat to life-safety in the event of an earthquake.

Figure 7 presents a histogram showing the number of fire stations built within certain decades. The advanced age of the stock of fire stations can be clearly seen since a significant number were built prior to the turn of the century. It is interesting to note that only 17 of the fire stations were built after the inclusion of seismic provisions in the Massachusetts building code. Most of the fire stations were built in

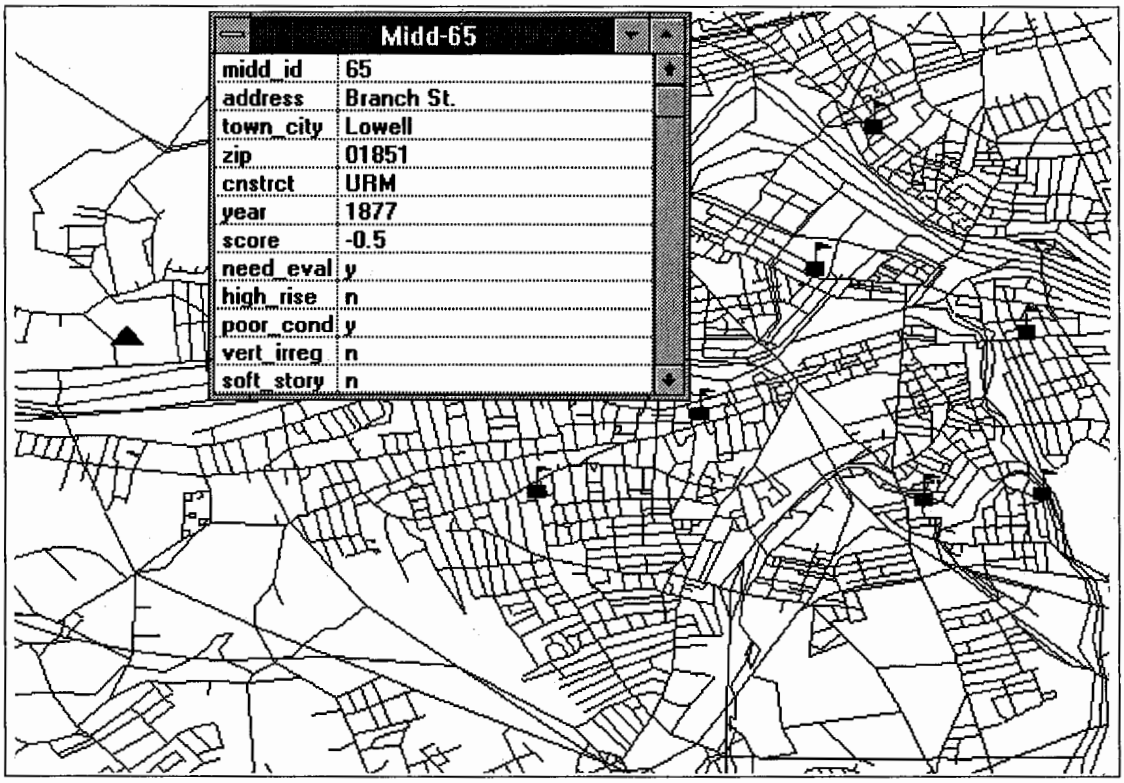


FIGURE 5. Response to selecting a fire station screen icon.

the 1950s and 1960s, before the inclusion of seismic provisions in 1975. A breakdown of the construction types found is shown in Figure 8. Unreinforced masonry construction, which has repeatedly demonstrated poor performance in past earthquakes throughout the country, makes up a clear majority of the fire stations.

Detailed Evaluation of the Branch Street Station

FEMA-154 recommends that structures receiving low scores in the visual screening should be subjected to a more detailed seismic evaluation. As noted earlier, many of the fire stations received low scores. However, conducting in-depth evaluations of each would not only be time consuming but also currently impractical. Therefore, one building possessing typical characteristics of the majority of those that "failed" screening was evaluated. The building chosen for this evaluation was the city of Lowell Fire Department's Branch Street Station No. 2 (shown in Figure 3) which was built in 1877. This building was singled out only for convenience — fire stations of similar construction and condition exist throughout Middlesex County, and probably the state.

Characteristics of this building that are typical of the others that received low screening scores were: unreinforced masonry bearing wall construction, plan irregularity, flexible wooden floor diaphragms, relatively tall hose tower, large openings along one wall and many non-structural falling hazards.

The evaluation of this building was conducted according to guidelines set forth in "A Handbook for the Seismic Evaluation of Existing Buildings" (FEMA-178 [ATC-22]).[6] This seismic evaluation is intended to determine the specific deficiencies in a structure's lateral force-resisting system. Once these deficiencies are identified, the engineer and owner must examine cost-efficient means of reducing the life-safety hazard of the structure. This evaluation typically involves examining the construction drawings and conducting an in-depth physical investigation of the building. Thus, this type of evaluation requires a large effort.

Midd-65	
midd_id	65
address	Branch St.
town_city	Lowell
zip	01851
cnstrct	URM
year	1877
score	-0.5
need_eval	y
high_rise	n
poor_cond	y
vert_irreg	n
soft_story	n
torsion	y
plan_irreg	y
pounding	n
lg_hvy_cld	n
short_clmn	n
pst_bnc_yr	n
floor_area	7600
use	Emer.Serv.
occupancy	0-10
nstrct_haz	y
comment_1	URM on right side of building
comment_2	2 different constructions (front and back) both are URM
comment_3	Floor to wall anchorage is provided, however not sufficient to protect
comment_4	from seismic loading (too small + sparse)
comment_5	
comment_6	

FIGURE 6. Data displayed for the selected fire station.

In accordance with FEMA's guidelines, evaluation forms pertaining to the building construction type (unreinforced masonry) were completed. These forms consist of statements that attempt to address life-safety issues. The statements are broken into various categories such as: building systems, masonry walls, diaphragms, connections, etc. Each statement is intended to lead the evaluator through a series of investigations and calculations in order to determine the safety of the building. Statements that evoke a true response are deemed acceptable according to the evaluation procedure. A false response indicates the need for further investigation. A response of "NA" signifies that this criterion is not applicable to this specific structure, and a response of "NC" denotes that this criterion was not considered as part of the evaluation.

Below are listed explanations of why certain evaluation statements were noted as false. Statements such as these must be remedied in order to reduce the life-safety threat. In order to clarify the description of the deficiencies of the evaluated building, the transverse direction is taken as the short direction of the building.

Weak Story. A deficiency was noted in the strength of the first-floor walls. Since both the first-floor walls and the second-floor walls were of the same thickness and construction, they possess the same unit shear strength. The ratio of strengths between the two stories is essentially the ratio between areas resisting shear in each story. It was shown that in the transverse direction, the strength of the lower story was 74 percent that of the second, less than the allowable difference of 80 percent. This deficiency is due mostly to the presence of the large garage door openings at one end of the structure.

Torsion. A severe torsion problem exists within the structure. There is an eccentricity between the center of gravity of the building and the center of rigidity of the lateral force resisting system. This eccentricity was shown to be 23.86 feet, more than the maximum allowable 20 percent of the width of

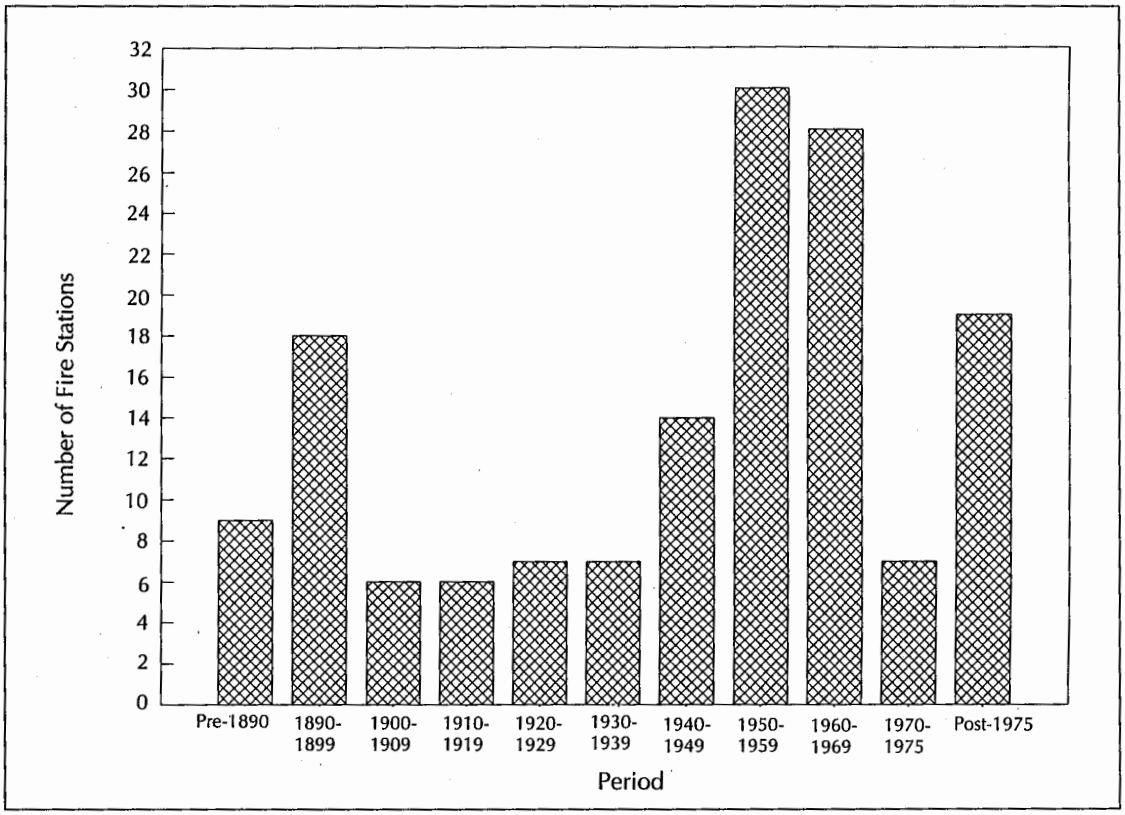


FIGURE 7. Distribution of construction dates among fire stations in Middlesex County.

the structure. Due to the large open front of the building, the center of rigidity was drastically shifted towards the rear of the building. When a lateral load is applied to the structure in the transverse direction, a torsional moment occurs. This moment causes a torque, or "twist," of the entire building.

Masonry Units. It was seen through visual inspection that much of the masonry was cracked and deteriorated. The deficiency is in the strength of the unit. No attempt was made to determine the reduced capacities of these masonry units since that would involve destructive testing.

Masonry Joints. Mortar could easily be scraped away from the joints with a key. Soft mortar has been shown to have reduced shear and bond strength. In some cases, the presence of mortar in the joints in the outer wythe of bricks was non-existent. Destructive tests would be required to determine the in-plane shear strength of the walls.

Shearing Stress Check. The average shearing stresses obtained through the "quick check" procedure[6] were in excess of the allowable value of ten pounds per square inch (psi), a conservative value based on the observed strengths of actual buildings with similar construction. The values obtained in the transverse direction were two to four times this allowable value, while those in the longitudinal direction were twice this value. A more detailed analysis was not conducted, realizing that shearing stresses resulting from torsion (which was not included in the "quick check") will increase the level of shear in the walls more than a detailed analysis will decrease it.

Plan Irregularities. The irregular plan of the structure poses a problem. When exposed to a lateral load in the longitudinal direction, the floor diaphragm will deflect as shown in Figure 9. At the re-entrant corners of the building (the circled regions), the deflection of diaphragm 1 does not coincide with that of diaphragm 2. Unless the dia-

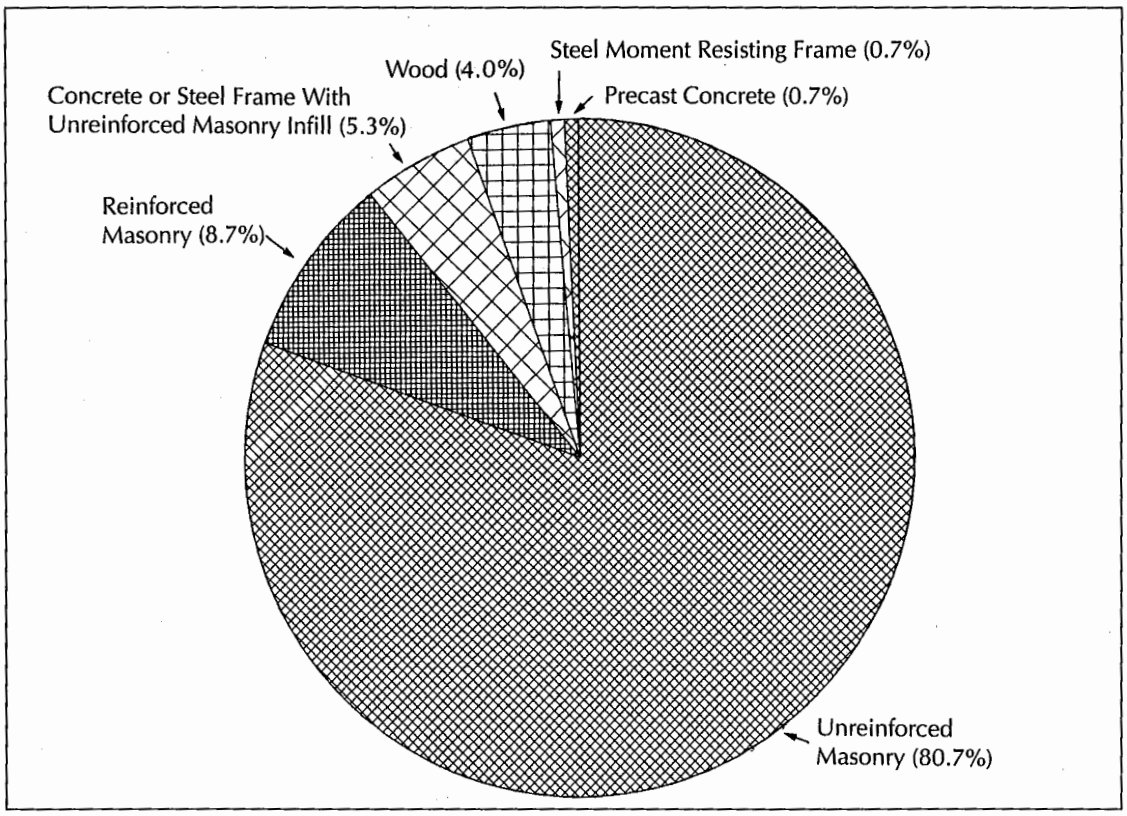


FIGURE 8. Distribution of construction types for fire stations in Middlesex County.

phragm sections have significant tensile capacity to resist their pulling apart in these areas, they will tear. The deficiency is in the strength of the diaphragm in the area of the corners.

Masonry Wall Anchors. Since finish carpentry covered the diaphragm to wall connections, the strength of those connections (if there is any) is unknown. Three built-up steel beams supporting the second level floor were attached to the wall with anchors (noticed from the exterior). The cover plates on these anchors are clearly too small to maintain stability of the normal walls during a transverse loading.

Anchor Spacing. The wall anchors present are spaced at approximately 13 feet. This spacing is far in excess of the four feet recommended in the FEMA guidelines. This lack of sufficient anchors can lead to partial collapse of normal walls when exposed to a transverse loading.

Lateral Support. The gable walls at the building ends could be laterally unstable. Height to thickness ratios of these gable walls is approximately 14:1, assuming the walls are one foot thick. Since no access was gained to the attic, the nature of how (or even if) these walls are laterally supported is unknown.

Anchorage. The gable walls and elaborate cornices present along the building's roof line are considered to be insufficiently anchored to the structural system. Again, since access to the connections (or absence thereof noted) was not gained, confident evaluations could not be made.

Unreinforced Chimneys. Two unreinforced chimneys, one of which is in very poor condition, extend above the roof level. These chimneys are non-structural falling hazards.

It is evident through this evaluation, that this building has numerous deficiencies in resisting lateral loads. The more critical of these deficiencies are a weak first story, torsion, shear strength and flexural strength of the walls.

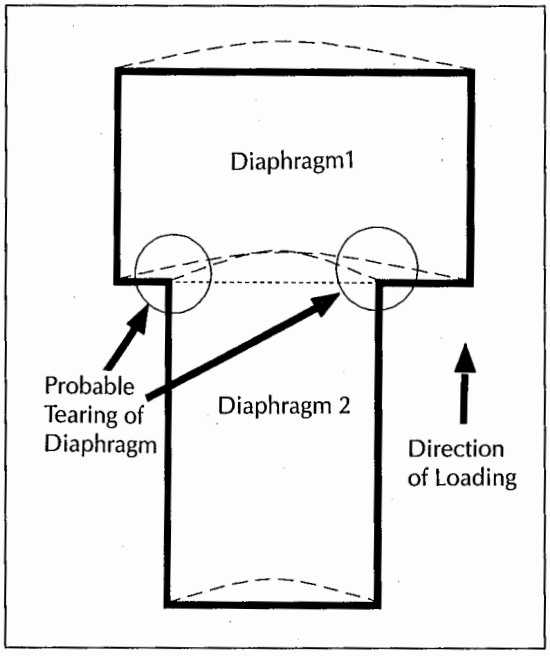


FIGURE 9. Diaphragm deflections at re-entrant corners.

These deficiencies could be remedied by the following:

- Tying a moment resisting steel frame to the inside of the front wall (and floor diaphragms) to resist lateral loads in the transverse direction. By doing this, the stiffness and shear strength of the lower story will increase, eliminating the weak story and shear stress deficiency, and the center of rigidity of the structure will shift forward, thus eliminating the torsion problem.
- Strengthening the floor diaphragms in the re-entrant corners, thereby eliminating the plan irregularity problem.
- Strengthening diaphragm-to-wall connections by adding more closely spaced and larger masonry wall anchors, thereby stabilizing walls with large height to thickness ratios and providing a continuous load path to the foundation.
- Strengthening the walls. One possible system is described in the section below. It involves bonding reinforcing to the wall surfaces. This system improves both the in-plane and out-of-plane strength of the walls.
- Bracing any gable walls and parapets. The bonding method mentioned just above would be applicable. This alternative would not only be effective, but also be cost efficient to building owners in the Northeast.

An Alternative Wall Strengthening System

Although many wall strengthening techniques are currently practiced in the Western states, their cost would seem unjustifiable in the minds of many building owners in the East. Methods such as bonding steel reinforcement to walls with shotcrete tend to physically detract from the appearance of the masonry, add considerable mass to a building and are stronger than what is probably justifiable in the East.

A lighter reinforcing system that would be cost effective, less obtrusive and still maintain the structural integrity of a building in the event of an earthquake needs to be developed. One such system could be the bonding of thin polymer films or filaments with epoxy to the face of the masonry wall. This concept grew out of the laboratory testing of plastics that demonstrated potential for high strength and large ductility, and a subsequent literature review on the use of polymers and fiber-reinforced plastics.

In order to determine whether the process of externally bonding reinforcement to a building's walls will eliminate their stability deficiency, the out-of-plane bending moments and tensile force resultants within the walls were evaluated. Different types of reinforcement materials were then investigated to determine whether they were capable of resisting these tensile resultants.

The out-of-plane lateral forces on each individual pier in the first story of the fire station were calculated according to the following UBC equation:[5]

$$F_p = ZIC_pW_p$$

Where:

F_p = The total lateral seismic force on an element in lbs

Z = Seismic zone coefficient, 0.15 for Massachusetts (Zone 2A)

I = Importance factor, 1.25 for an essential structure

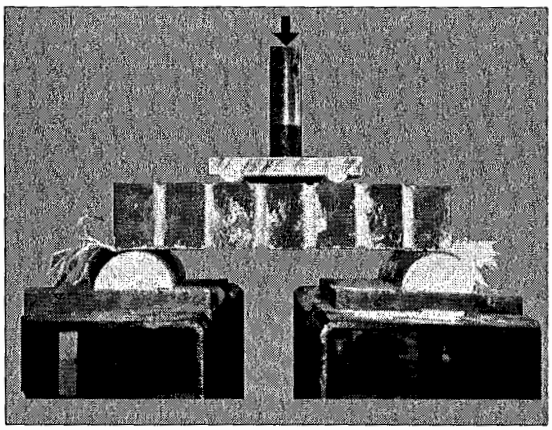


FIGURE 10. Flexural beam test setup.

TABLE 1
Flexural Beam Test Results

Reinforcement	Load (lbs)	F_g (psi)
Polyvinylbutyral	467.29	74.52
Glass Fiber—2-in. Mesh	1,925.86	293.31
Glass Fiber—2.67-in. Mesh	1,887.85	287.61
Carbon Fiber	705.92	110.30
Monofilament	614.13	96.55
Plastic Straps	335.20	54.71
Unreinforced	292.21	48.26

C_p = Horizontal force factor equal to 0.75
W_p = Weight of element or component (lbs) is equal to 120 lbs/ft^2 times the effective load width times height (effective load width is the width of wall whose weight is tributary to the pier in question)

Conservatively assuming each pier to act as a uniformly loaded simple span beam, the bending moments caused by these forces were determined to range from 10,283 to 132,891 in-lbs, with an average of 39,868 in-lbs (excluding the pier between the large door openings). Next, assuming that the masonry itself possessed no tensile strength, the resulting tensile force that was assumed to act at the face of the wall (where the reinforcement would be applied) was determined for each pier. The calculated forces ranged from 27.5 to 71.4 lb/in, with an average of 59.7 lb/in. Any acceptable reinforcing scheme must be capable of developing these forces.

An experimental program was developed to determine whether the external bonding of reinforcing materials would increase the flexural and diagonal tensile strength of unreinforced masonry walls. The materials studied were polyvinylbutyral film, glass fiber, carbon fiber, monofilament fishing line and plastic packaging straps. These materials were chosen mainly because they were readily available and/or inexpensive, added only 0.03 inch to the thickness of the walls (and correspondingly little weight), and could be applied to a building while it was still in use. These alternatives were bonded to the face of the masonry with a two-part epoxy.

Flexural beam tests were performed to determine the effect of the reinforcing on the flexural strength of the masonry. The flexural beam test was used because of its versatility and ability to produce data with little effort and cost. This test is, potentially, a simple and economical representation of the behavior of walls subjected to out-of-plane lateral loadings. The flexural beam test was conducted in accordance with ASTM E518-80.[7] The specimen was placed horizontally across a 15.75-inch span so that the reinforced face would be placed in tension (see Figure 10). A third point load was applied to the specimen manually with a hydraulic actuator until failure.

The gross area flexural tensile bond strength of the specimens was calculated using:[7]

$$F_g = ((P + 0.75P_s)L)/bd^2$$

Where:
F_g = Gross area flexural tensile bond strength
P = Maximum total applied load
P_s = Weight of the flexural beam specimen
L = Span length
b = Average width of the specimen
d = Average depth of the specimen

Average values of applied load and gross area flexural tensile strengths obtained from testing the flexural beams are presented in Table 1.

An average flexural bond strength of 74.52 psi was observed for the polyvinylbutyral film reinforced specimens. Although more signifi-

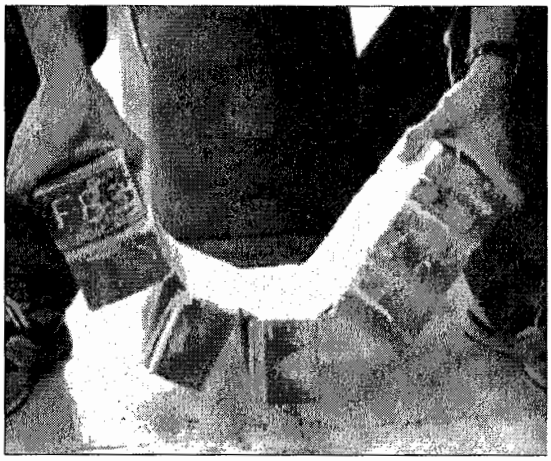


FIGURE 11. Failed polyvinylbutyral reinforced specimen.

cant increases in strength were observed with other reinforcements, the great post-cracking ductility imparted by the film is worth special note. The specimen failed successively in three joints. It was obvious that all the tensile force in the specimen was being developed in the film. Even though complete failure of the mortar to brick bond was observed, the polyvinylbutyral film held the specimen intact, as illustrated in Figure 11.

Flexural cracks were observed in the mortar joints of the two-inch-spaced glass fiber reinforced specimens at a load level of approximately 300 lbs, corresponding to 49 psi. These specimens ultimately failed in shear at their first joint before failing in bending at midspan. Even though less reinforcement was present in the 2.67-inch-spaced glass fiber reinforced specimens, their flexural bond strength was approximately the same as that of the two-inch-spaced specimens. Since all glass fiber reinforced specimens failed in shear outside of the middle third of the span, it was concluded that the amount of reinforcing had little effect on the strength of the beam. It was felt that the masonry failed in shear before the true flexural capacity of the specimens was obtained.

As the shear and moment diagrams in Figure 12 indicate, failure within the middle third of the beam would be caused by bending stresses. Outside this region, the beam is exposed to both shear and bending. The applied load could cause a shear failure in the region before the ultimate moment capacity of the beam is reached in the middle section.

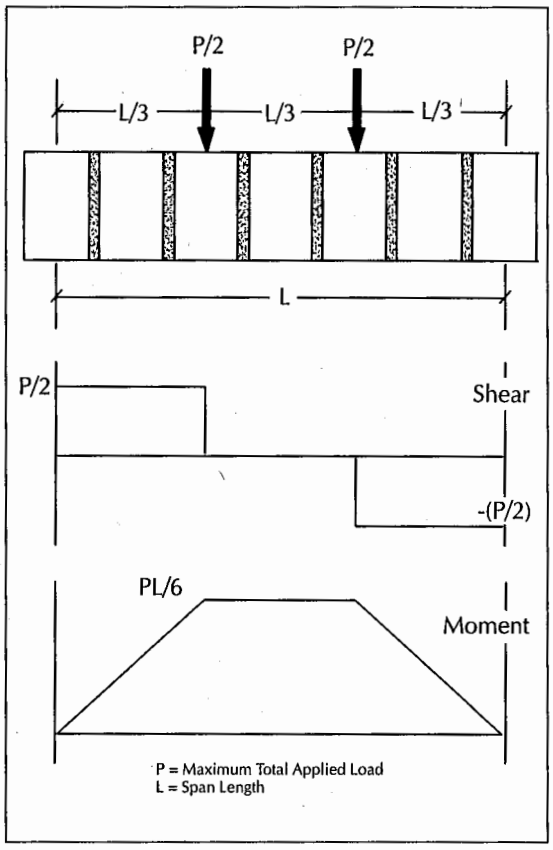


FIGURE 12. Shear and bending moment diagrams of third point loading.

To determine if this was the case for the glass fiber reinforced specimens, the shear strength of the masonry was estimated and compared to the actual shear stress in the specimens at failure. In place of actual testing, Brick Institute of America formulas were utilized in estimating the shear strength of the masonry. Based on the compressive strength of the brick of 10,940 psi (obtained from the manufacturer), the compressive strength of the masonry, f'_m, was determined to be 1,727 psi.

The shear strength of the masonry was then taken as:[8]

$$f_v = \frac{0.5\sqrt{f'_m}}{0.75}$$

Where:

f_v = The shear strength of the masonry (psi)

f'_m = The compressive strength of the masonry (psi)

0.75 = Factor to increase value from the ultimate stress level to the working stress level

The shear strength and corresponding load to cause shear failure were determined to be 27.7 psi and 1,558 lbs, respectively. The experimental results of the glass fiber reinforced specimens indicated applied loads ranging from 1,875 to 1,976 lbs. It is obvious that the shear strength of the specimens was exceeded, thus inducing failure.

The average flexural bond strength of the carbon fiber reinforced specimens was determined to be 110.3 psi. Brittle failure of the specimen was observed within the interior third of the span.

Neither the monofilament line nor the plastic strap reinforced specimens were able to develop their ultimate strengths. Poor bonding between the reinforcement and the masonry allowed the material to "peel" from the brick. If better bonding agents are discovered, these two materials could prove effective.

The increase in strength due to the various reinforcing methods ranged from 37 to 537 percent. It is obvious that the glass fiber showed the greatest increase in flexural bond strength. Although the polyvinylbutyral reinforcing increased the strength only 66 percent, it was able to contain the masonry after failure (as was the glass fiber reinforcing).

An analysis was conducted to convert the tensile forces in unreinforced masonry walls due to out-of-plane bending (calculated for the fire station earlier) to a flexural beam load that would produce the same magnitude of force. Once calculated, these flexural beam loads were compared to those obtained through testing of the reinforced specimens, to determine which, if any, reinforcement scheme(s) will produce the flexural capacity needed for seismic retrofit.

A free body diagram illustrating what occurs at failure in the beam specimen due to third point loading is presented in Figure 13. In order to simplify the analysis, the effects of the self-weight of the specimen as well as the weight of the loading attachment were neglected. Also, the mortar was assumed to have no tensile strength, a commonly made assumption in the analysis of masonry structures.

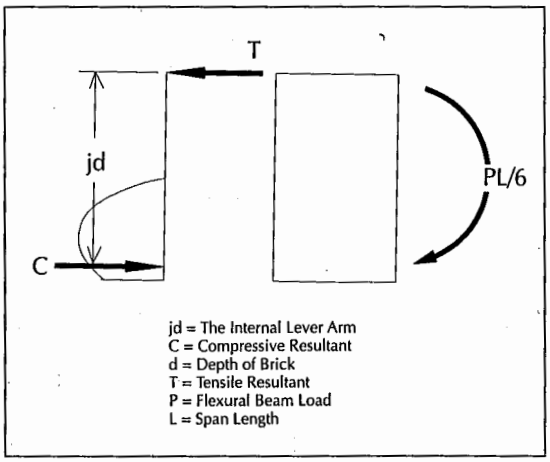


FIGURE 13. Free body diagram of a flexural beam specimen.

An equilibrium condition was obtained by summing moments about the compressive resultant, *C*. Assuming that $j=1$, the flexural beam load, *P*, required to produce the resultant tensile force in the beam, *T*, was calculated using:

$$P = (6Td)/L$$

Where:

P = Equivalent flexural beam load, lbs
T = Width of flexural beam (7.5) times the out-of-plane bending force, lbs per inch calculated earlier
L = Span length of the flexural beam, inch
d = Depth of the brick specimen, inch

These calculated flexural beam loads ranged from 275.4 to 765 lbs. A value of 1,960.7 lbs (corresponding to the load level in the pier between the large door openings) was not considered to be critical. If a complete retrofit of this structure were undertaken, a different strengthening procedure, such as the implementation of a steel frame, could and probably would be used to retrofit the wall containing this pier.

As seen in Table 1, the reinforcing of specimens with glass fiber spaced at both two inches and 2.67 inches, with flexural beam loads ranging from 1,875 to 1,976 lbs, were the only alternatives that were adequate to develop the required strengths with a factor of safety on the order of two.

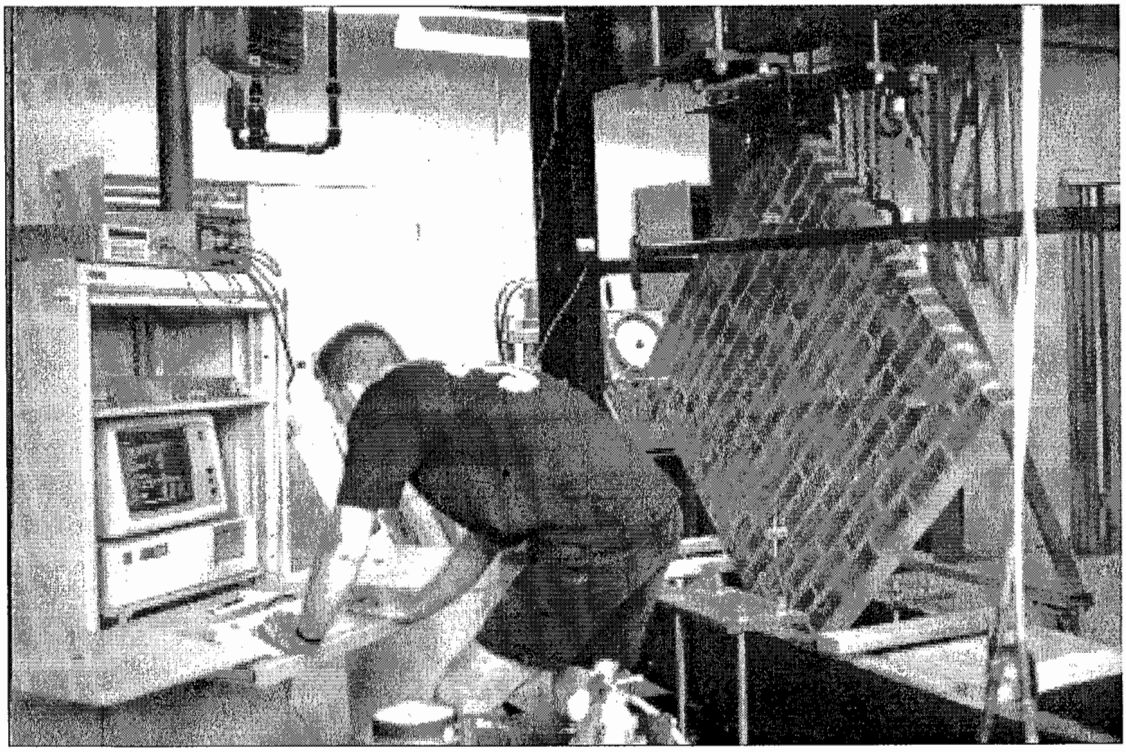


FIGURE 14. Diagonal tension test setup.

The flexural beam tests addressed walls subjected to out-of-plane loading. Diagonal tension tests were also conducted to investigate the resistance of walls to in-plane loading. It is a deficiency in a combination of this diagonal tensile strength and shear along horizontal bed joints that leads to the failure of a wall exposed to an in-plane lateral loading.

These tests, shown in Figure 14, were conducted in accordance with specifications set forth in ASTM E519, where a four-foot square panel is loaded to failure along one of its diagonals.[9] Tests were performed on four test panels — two unreinforced, one reinforced with a two-inch glass fiber mesh (laid up by hand with glass fiber roving) and one reinforced with polyvinylbutyral film. As a result of previous flexural testing, the glass fiber reinforcement was chosen for its strength, while the polyvinylbutyral film was chosen for its ductility.

A common interpretation of the diagonal tension test results was made. The average shear stress at diagonal tension failure, considered to be the component of the applied load parallel to the bed joint divided by the cross-sectional area of the bed joint, was calculated. Table 2 contains the values of the load and the average shear stress corresponding to failure.

Upon failure of the unreinforced specimens, each "exploded" into two large pieces with a loud noise. A large difference in ultimate strength was observed between these two specimens. Since each was constructed by a different person, one an experienced mason and the other a novice, this difference in strength was attributed to a difference in workmanship (the stronger of the specimens was constructed by the experienced mason). This assumption lends support to the significance of the masonry joint deficiency noted earlier.

Failure of the glass fiber reinforced specimen, also accompanied by a loud crunching noise, was initiated at the center of the panel and propagated along the vertical diagonal. Despite the fact that some of the glass fiber broke across the plane of the crack, enough fiber remained intact to hold the failed pieces of the panel together. It is also interesting to note that as load was applied to the panel, cracks became clearly visible on its unreinforced side.

No cracks became visible on the reinforced side of the panel until failure was observed. This differential movement between the two sides of the panel caused an outward bow or bend in the wall. Ultimately, if this system is used as a retrofit technique, the reinforcing may be put on both faces of the wall to support the tension developed on the two faces due to the cyclic nature of seismically induced loads.

Failure of the polyvinylbutyral reinforced panel, similar to that of the unreinforced specimens, initiated at the lower loading shoe and propagated upward along a bed joint. Since failure did not initiate in the center of the panel, it is hypothesized that failure was not due to diagonal tension, but rather to bed joint shear failure, originating at a joint where the load shoe bears on the panel. At failure, the polyvinylbutyral film was able to hold the broken portions of the wall together as a single unit. Although the film was in a plastic state at the time, it still managed to prevent the total collapse of the panel.

Both reinforcement methods were seen to increase the shear capacity of the masonry. When compared to the required increase in shear necessary for retrofit of the evaluated building (60 psi — calculated to be 1.5 times the average shear stress in the walls) both reinforcing schemes were found to be adequate.

TABLE 2
Diagonal Tension Test Results

Reinforcement	Load (lbs)	Shear Stress (psi)
Unreinforced	4,835	18.99
Unreinforced	21,478	84.36
Glass Fiber	24,325	95.50
Polyvinylbutyral	24,525	96.33

The Future

Taking into consideration the age of most of the fire stations that were screened, the limited maintenance that many of them have received, and the fact that 121 of them are constructed of unreinforced masonry, it can be concluded that 80 percent of the fire stations in Middlesex County are likely to be damaged in the event of a significant earthquake.

As shown through the experimental program, externally bonded reinforcement in conjunction with conventional bracing and anchoring could be utilized to strengthen the unreinforced masonry fire stations, as well as other unreinforced masonry buildings. An added observed feature of some of the bonded reinforcements was their ability to contain the bricks after failure. Since falling bricks pose a large threat to life-safety and property, this is a beneficial attribute. Containment of the bricks also results in a higher level of overall building integrity, therefore allowing safer evacuation of the occupants of damaged buildings.

Due to limitations of both the flexural beam and diagonal tension test procedures, further testing should be done prior to adopting the bonded reinforcing system. In the flexural beam tests, shear failures were sometimes occurring before flexural failures, which resulted in underestimating possible increases in the flexural strength of retrofitted walls. In the case of the diagonal tension tests, it is doubtful that the test is a good representation of the actual behavior of a wall subjected to seismically induced in-plane loads. In both cases, larger specimens with more realistic boundary conditions and loading are needed.

The practice of rapid visual screening of essential facilities in Massachusetts continues. Funding has been obtained from the Massachusetts Emergency Management Agency to continue this work during the summer of 1993. A broader range of essential buildings over a larger geographic region are being examined. Schools, hospitals, fire stations, emergency operating centers and emergency shelters in a distribution of communities in Middlesex, Norfolk, Essex and Suffolk counties are being screened, with an estimated total number of structures being examined as high as 300. Even though that is a relatively small number compared to the total number of essential buildings in the state (which is estimated to be in excess of 3,500), valuable information on the general character of these buildings is being gathered.

Once collected, these data will be reviewed with the goal of identifying common problems among the different types of essential buildings. With or without common problems, cost

studies for retrofitting "typical" essential buildings could begin using the data from the screenings as a planning tool for that study.

This work represents an initial effort. Much more work needs to be done. It is hoped that this study will help promote further work towards reducing seismic risk in Massachusetts.

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STEVEN P. MCELLIGOTT *is a Civil Engineer working in the Transit Systems Engineering department of R.W. Beck and Associates in Framingham, Massachusetts. He joined the firm in 1992 and specializes in the design of overhead contact systems. He has previously worked as a civil engineering technician for the U.S. Army Corps of Engineers in Waltham, Massachusetts. He received his B.S. and M.S. degrees in civil engineering from the University of Massachusetts Lowell, where his graduate work focused on seismic evaluation and retrofit of unreinforced masonry buildings.*

JAMES R. GAGNON *is a graduate student at Rensselaer Polytechnic Institute where, under a full fellowship, he is studying environmental engineering. He received his bachelor degree in civil engineering from the University of Massachusetts Lowell. He is a Certified Engineer-in-Training in Massachusetts and a student member of ASCE.*

CHRISTOPHER H. CONLEY *is an Assistant Professor in Civil Engineering at the University of Massachusetts Lowell where he has been since 1989. Prior to coming to Lowell, he was a Senior Research Associate at Cornell University for two years and a Member of Technical Staff at Sandia National Laboratories in Albuquerque, New Mexico for four years. He has earned an A.S. degree in Civil Engineering Technology from Greenfield Community College in Greenfield, Massachusetts, a B.S. degree in Civil Engineering from the University of Massachusetts at Amherst, and M.S. and Ph.D. degrees in Civil Engineering from Cornell University.*

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