CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

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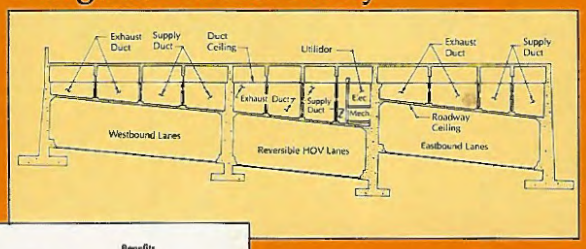
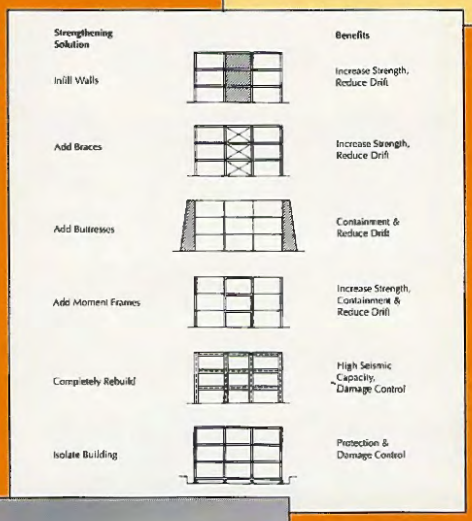
Design for Tunnel Safety



Seismic Strengthening of Buildings



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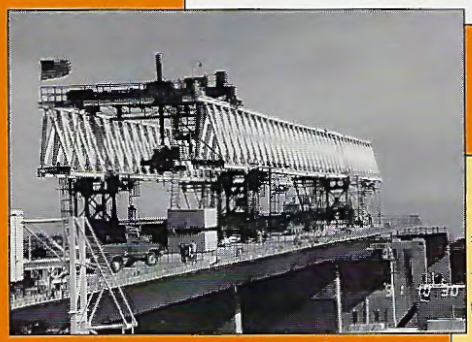
- Using a Traffic Forecasting Model for a Major Highway Project
- The Charles River Basin



Launching Gantries for Bridge Erection

Groundwater Conditions During a Deep Excavation

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

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

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Practice Can Make Perfect

There has been a perceived gap of what is available in civil engineering journals and publications. Many journals publish very theoretical papers that are not directly applicable for practicing engineers. The Boston Society of Civil Engineers (BSCE) began publishing its own journal in 1914. This journal focused on papers presented before the Society, the accomplishments of members, reports of professional committees, items of general interest and Society proceedings. Ten years ago the Board of Government of the Boston Society of Civil Engineers Section/ASCE charged a select committee to "make a comprehensive evaluation of the [Society's] Journal, taking into account all pertinent factors — financial, editorial, advertising, readership and production." This committee recommended many changes to the Society's Board of Government, including an abiding emphasis on papers that would have practical use for engineers, as well as a change of name to reflect that emphasis, and changes in appearance and format.

Over the past eight years, *Civil Engineering Practice* has published practice-oriented papers — case studies describing different projects, reviews of current practices and standards, discussions of practical engineering methods for analysis and design, examinations of the application of new procedures, historical perspectives, and other articles. What we have heard out in the field has been encouraging. Many engineers throughout the world have commented on the accessibility, quality and usefulness of the papers. Issues of *Civil Engineering Practice* have become valuable reference documents. Some articles have resulted in lively debate on practices.

The development and successful execution of *Civil Engineering Practice* in its current form can be attributed, in large part, to the efforts of one individual who wishes to remain anonymous. This past President of BSCES pretty much started the revised journal from scratch and has been primarily responsible for guiding it to its present form. This person's hard work, zeal and vision — along with the many people who have served on the journal's editorial board, soliciting and reviewing papers; others who have volunteered as peer reviewers of papers; and our contributers who have shared their insights — have made it possible for *Civil Engineering Practice* to fulfill its mission. Today, BSCES can be proud of the document it publishes. *Civil Engineering Practice* is unique, not only in its practice-oriented format, but also as a regional representation of civil engineering experience that really is not duplicated anywhere else in the country.

Civil Engineering Practice hopes to continue what it has done best — focusing on practical engineering topics. Articles will be oriented toward local work, with a mix of some national and international projects (see our call for papers on page 96). We hope to keep building on its strong foundation, recognizing and acknowledging the importance of practice.

Brian Brenner

Brian Brenner
Chair, Editorial Board

CIVIL ENGINEERING PRACTICE

Founded in 1848, and having merged with the American Society of Civil Engineers in 1974, the Boston Society of Civil Engineers is the oldest engineering society in the United States. Examination of the Society's history yields a proud record of involvement and leadership in both technical and professional endeavors within the civil engineering profession. Underlying the activities of the Society since its founding is a firm resolve to provide the means for civil engineers to communicate advances in the art and science of civil engineering. The creation of the Journal in 1914 resulted from that resolve, and *Civil Engineering Practice* demonstrates our continued commitment.

In *Civil Engineering Practice* we are seeking to capture the spirit and substance of contemporary civil engineering practice in a careful selection of articles which are comprehensive in scope while remaining readily understandable to the non-specialist. Typically using a case-study approach, *Civil Engineering Practice* places key emphasis on the presentation of techniques being applied successfully in the analysis, justification, design, construction, operation and maintenance of civil engineering works.

Seismic Strengthening of Existing Buildings

The methods of earthquake damage mitigation — including strengthening buildings and reducing earthquake forces — show a high degree of variation from building to building.

NICHOLAS F. FORELL

The strengthening of existing buildings to resist earthquake forces has become over the years an issue of increasing concern and activity not only of engineers but also of the design profession as a whole, including governmental regulators, developers and real estate users. The reason for this concern is a growing awareness of the hazards of earthquakes and the governmental response to those hazards. Earthquakes pose not only a serious threat to life, but also the economic consequences of even a moderate earthquake can be disastrous. Not only can the repairs of buildings and infrastructure be very expensive, but also the cost of the disruption of business and services can have a long-lasting detrimental effect. Governments, both on the local and federal level, are ill-prepared to provide adequate financial aid to stricken communities. Clearly, an approach that addresses the causes of earthquake disasters rather than its effects is in the national interest. It is more cost effective to prevent a large-scale disaster through an earthquake awareness program that leads to the strengthening of hazardous buildings and facilities than to financially support the reconstruction of destroyed or damaged buildings or infrastructure.

This awareness led to the passage of the Earthquake Hazard Reduction Act of 1977 (Public Law 95-124), which directs the President "to establish and maintain an effective earthquake hazard reduction program."[1] The strengthening of existing seismically hazardous buildings is clearly an important element of earthquake hazard reduction.

An earthquake hazards mitigation program has three cornerstones:

- The establishment of the seismic risk;
- The determination of the vulnerability of structures; and,
- The development of hazard mitigating building codes and regulations.

Seismic Risk

The establishment of seismic risk is a complicated and often controversial task. Little action will be taken in earthquake hazard mitigation if the risk of earthquakes is not understood or not taken seriously. Even in California, where most people who have lived there for any length of time have experienced an earthquake, the full severity of a major earthquake does not

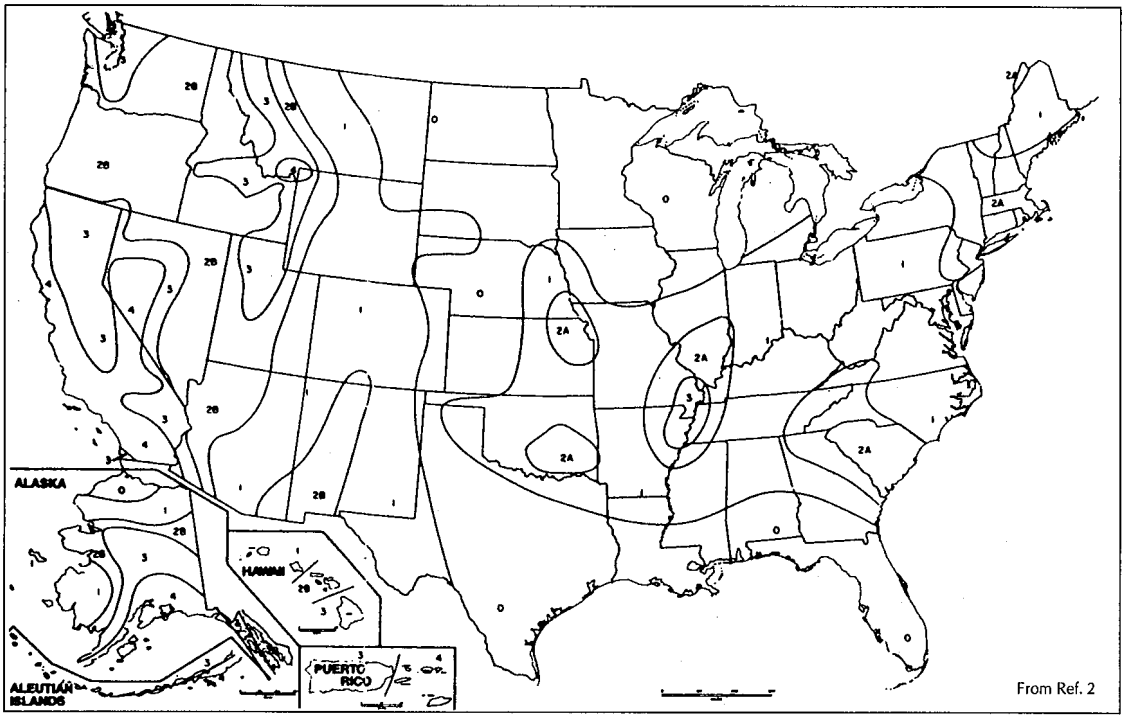


FIGURE 1. A UBC seismic zonation map of the United States.

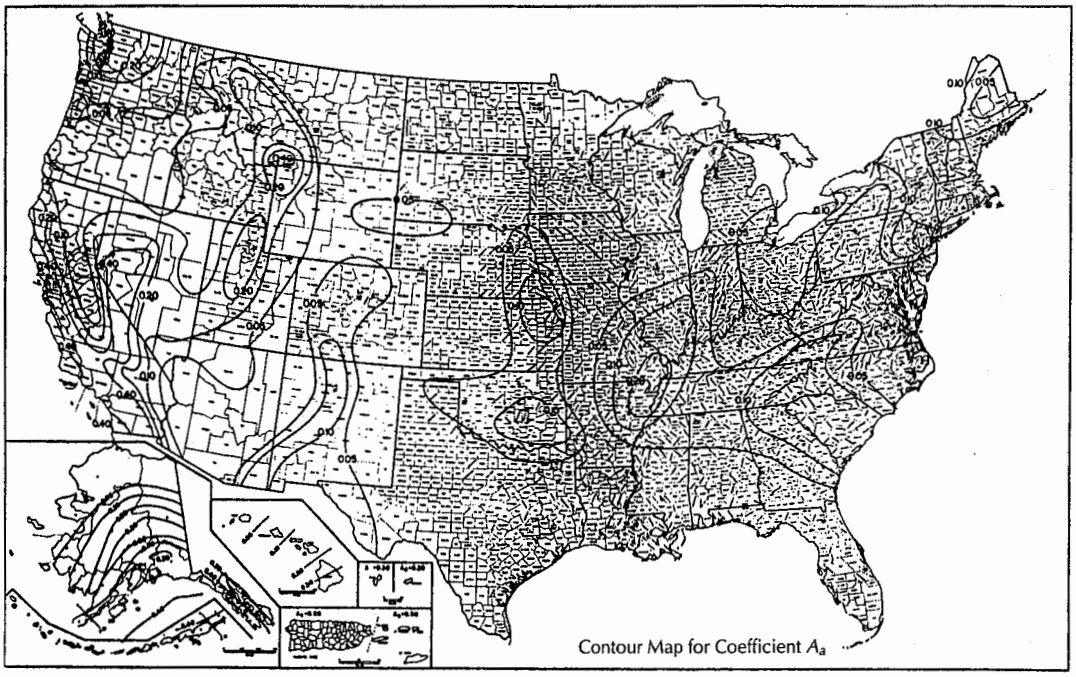
appear to be a driving force. In instances where there is sufficient awareness of the seismic risk, the inconvenience and cost of mitigating earthquake hazard in existing buildings override that awareness. It is therefore not surprising that seismic risk awareness and hazard mitigation is low in areas of lesser seismicity.

The seismic risk of an area is based on its past history of earthquake events as well as on geological observations and deductions. Regions of high seismicity naturally have quite extensive records, but in regions of low seismicity, such as New England, the historical record is sparse. Earthquakes were recorded only if they were of sufficient significance or left traces of their existence. Since ground motion recording devices are relatively recent inventions, quantitative data on historic earthquakes have been matters of deduction. In addition to historic records, seismicity can be determined by geological and seismological exploration to locate existing faults, both active and inactive fault traces, and by the assessment of geologic and tectonic features.

It can be argued that even though the frequency of earthquakes in the New England-New York region may be considerably lower than in California, the seismic risk might well be higher.

The design assumptions behind the *Uniform Building Code* (UBC) [2] and the *National Earthquake Hazard Reduction Provisions* (NEHRP)[3] guidelines are based on a 475-year event, or an event with a ten percent probability of not being exceeded in 50 years. In the case of California, the difference between this design level and the maximum credible earthquake is not great. (The maximum credible earthquake is defined here as the largest earthquake that a particular fault mechanism can generate.) Modern well designed structures in California should survive such an event without collapse even though they may suffer considerable damage.

In regions of low seismicity where very large earthquakes can occur (although they have a low probability of occurrence), the difference between the current design earthquake and the maximum credible event can be very large. Therefore, the rare but possible event, which may have a 1,000-year occurrence, can have a devastating impact on the region.



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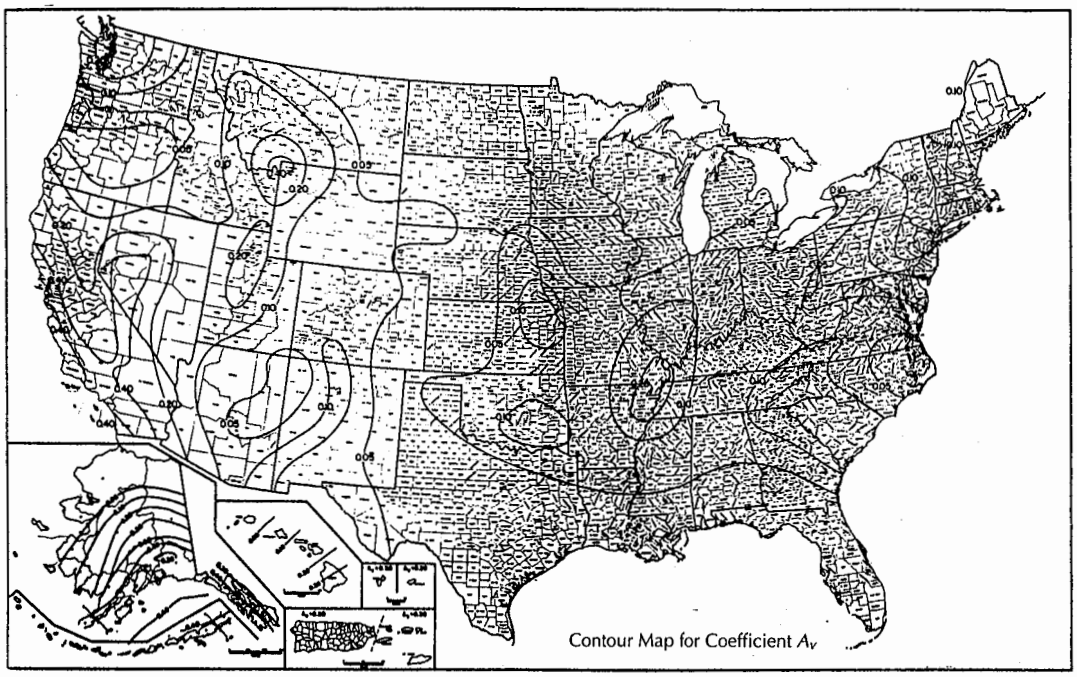

FIGURE 2. NEHRP seismic zonation maps of the United States.

Seismic zonation is an ongoing effort. Currently used zonation maps are included in the UBC (see Figure 1) and NEHRP (see Figure 2). The NEHRP zonation maps are currently under revision (see Figure 3). Earthquake-resistant design is based on an assumption of the seismicity and seismic risk of a specific region. Estimates based on recorded earthquakes carry

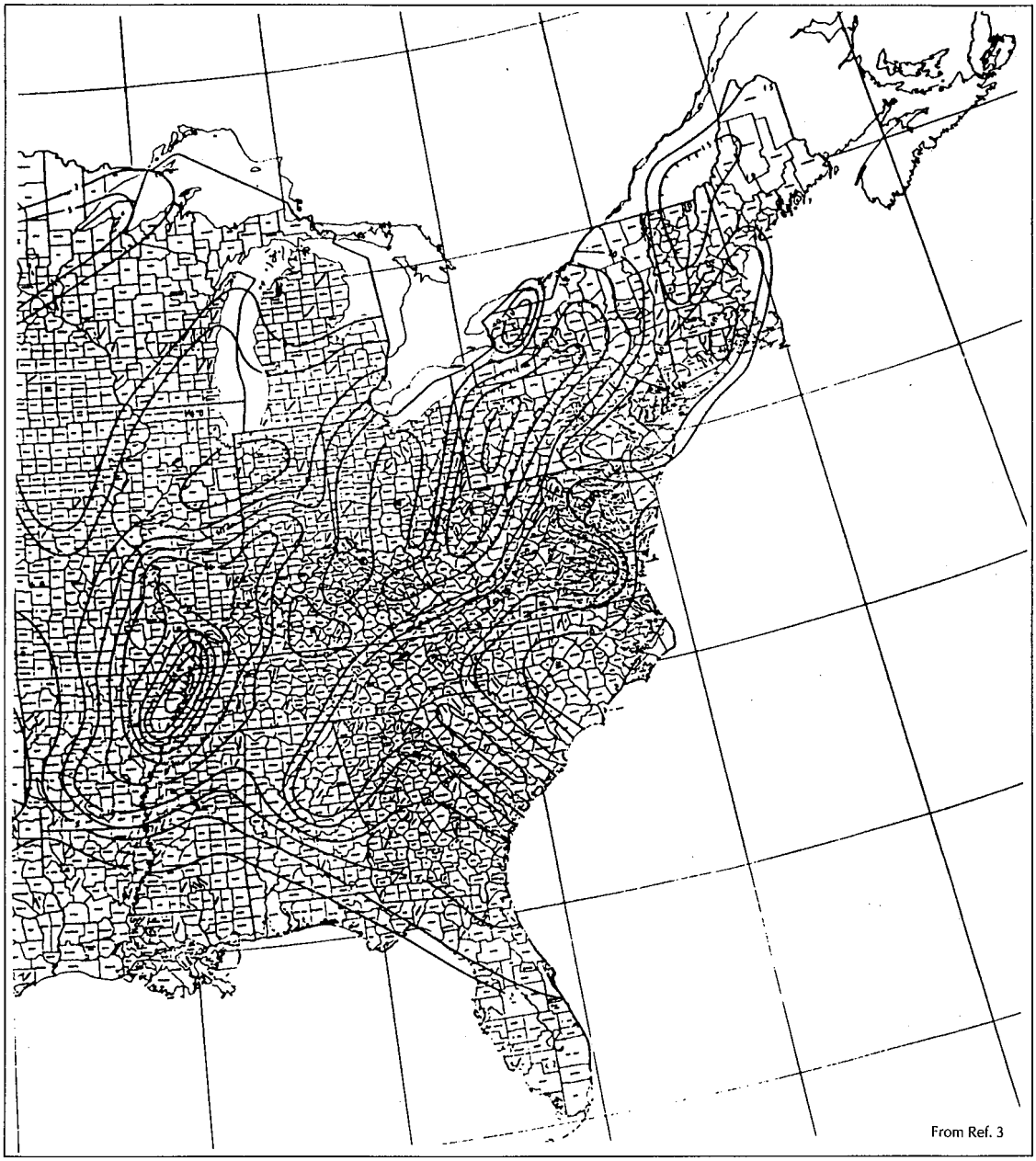

FIGURE 3. Proposed new NEHRP seismic zonation map of the eastern United States.

with them a degree of uncertainty that depends on the completeness of the earthquake catalogs used, the time span of the catalog (which may be less than the average interval between the larger earthquakes) and the lack of knowledge of the magnitude of historic earthquakes. The determination of seismic risk should be given the highest priority in a hazard abatement program. The engineer must know the demand that earthquake forces can place on a structure in order to provide the appropriate design capacity of the structure to resist the demand.

Vulnerability of Structures

Seismic vulnerability can be determined by a careful evaluation of the structural system and the detailing used. Examples of such an examination are:

- The identification of the lateral load resisting elements, their location and distribution, as well as their consistency over the height of the structure.
- The detailing of the structural elements and their ability to transmit lateral forces to the resisting elements.
- The compatibility of cladding and finishes with the building's anticipated behavior.
- Adjacency to other buildings and the possibility of pounding between structures.
- Signs of previous damage or deterioration of the structure or finishes.

An analysis should quantify the resistance capacity of the structure. An important part of the analysis is the application of a model that takes into consideration all structural elements that participate in the lateral resistance of the building. Elastic and inelastic properties must be taken into consideration. The importance of judgment developed by field observations of the effects of earthquakes on structures is not to be underrated. Much of what has been observed has become the basis of earthquake design provisions.

The program entitled "Learning from Earthquakes," sponsored by the Earthquake Engineering Research Institute (EERI) with the support of the National Seismic Foundation, has provided many engineers with the opportunity to observe and study firsthand the effects of earthquakes, and has provided a valuable library of field observations. Past earthquakes have provided a laboratory where engineers, architects, disaster response personnel and others concerned with the effects of earthquakes can see the failures and success of efforts to mitigate the effects of earthquakes.

Visiting a stricken community can leave an indelible impression. Field observations are of special value in assessing the vulnerability of buildings that were designed prior to the development of modern building codes and design and analysis procedures. The list of lessons learned via these observations is long, but a few salient examples are:

- The performance of unreinforced masonry is consistently poor.
- Walls need to be securely tied to floors and roofs (also true for tilt-up construction).
- Non-ductile concrete frames perform poorly and shear failures of beam-column joints are common.
- Reinforcing bars need to be adequately spliced and anchored.
- Flat slab construction, which has very little tolerance for story drift, performs poorly.
- The P-Δ effect of taller buildings must be considered in design.
- A quality assurance program should be implemented in the field.

By far, the greatest hazard can result from an inappropriate building configuration. Changes in stiffness over the height of the structure, soft stories with large openings at the ground floor, or unusually high stories are hazardous unless they are carefully designed. Re-entrant corners, such as in T-shaped or L-shaped plans, raise stresses and can lead to serious problems.

In evaluating the vulnerability of a building, the geotechnical setting must be considered. Obviously ground failure, settlement or liquefaction has a profound effect on the structure. The 1985 Mexico Earthquake, and to some degree the Loma Prieta Earthquake in the San Francisco Bay Area in 1989, have demonstrated that even distant earthquakes can, under certain soil conditions, amplify ground motions and make buildings in resonance with these motions extremely vulnerable.

A study of the vulnerability of structures to resist earthquakes is a key issue in successful earthquake engineering. It is critical to know where, how and why structures and structural elements fail and which are the weak links. Careful analysis can provide the answers, but the study of past events is invaluable in developing judgmental understanding.

Regulations & Voluntary Hazard Abatement

Building codes and governmental regulations regarding reducing the risk from seismically hazardous buildings fall into two categories: *active* hazard abatement and *passive* hazard abatement programs.

In active programs, a target is typically identified. This target may be a type of building,

such as unreinforced masonry buildings or building components such as parapets or elevators, which are judged to pose a hazard that requires corrective work. Examples of such active programs are the Los Angeles Unreinforced Masonry Strengthening Ordinance, the San Francisco Parapet Safety Program, the State of California Retroactive Elevator Requirements, and the State of California Unreinforced Masonry Abatement Program.

A common denominator of these programs is that time limits are established for compliance with the requirements that allows for progressive corrections in some cases. Generally, the requirements are significantly different from those required for new buildings since they are less stringent than those for new construction.

Passive hazard abatement programs are seismic strengthening requirements that are "triggered" by proposed changes to the building, such as an increase in occupancy load or hazard classification, alterations to the structural system, or alterations that affect a certain percentage of the area of a building or increase the value of a building. Compliance to the levels required for new construction may be difficult to accomplish and extremely expensive, therefore the requirements may be less stringent.

For example, Section 104f of the *San Francisco Building Code* (SFBC) requires the upgrading of a structure to only 75 percent of the code requirement for new buildings.[4] The rationale for that requirement is that at this level *life-safety* (a sufficient probability that persons in and around the structure will not be harmed) should be provided, even though a certain degree of property damage is accepted. The complexity of compliance frequently requires a case-by-case negotiation between the owner and the appropriate governmental agency.

The most common passive hazard abatement program method in codes has been what is called the "25-50 Percent Rule." Basically, this rule states that if the proposed work on a building exceeds 50 percent of its value, full compliance with the code is required. If the work is valued between 25 and 50 percent of the building value, the requirements are lessened and often pertain only to the alterations themselves. If the work is less than 25 percent of the building value, the work must not endanger public safety or extend existing hazards. The philosophy of this rule, which requires an increasing level of seismic upgrading with increasing levels of building alterations or additions, has found its way into many building codes.

Earthquake damage is another "trigger" that leads to mandatory corrective work. For example, in San Francisco, if the damage requiring repair affects more than 30 percent tributary to the vertical or lateral load carrying elements, or of these elements themselves, the entire structure of the building must be brought up to meet the current code requirements.

There is evidence that the amount of voluntary earthquake hazard abatement work is increasing. Although the motives may be manifold, the dominant ones appear to be to protect life-safety, to protect property and to maintain business activity. Examples can be found in the high-tech industry in Silicon Valley, south of San Francisco. IBM has strengthened buildings, braced piping systems, anchored equipment, and has established in-house seismic safety standards. Hewlett-Packard, Intel and Apple are further examples of many companies that have established active seismic hazard abatement programs. It can be conjectured that in this industry enlightened management, expensive equipment, difficult to replace research and intense competition have triggered this activity.

Utilities have also developed their own programs in seismically strengthening their facilities. The Pacific Gas & Electric Company, the East Bay Municipal Utility District, the Southern California Gas Company, and the Los Angeles Department of Water and Power are examples. Private building owners and real estate investors have also become more active in seismic hazard abatement.

Although most of this activity is taking place in California, there are examples of voluntary efforts in other states. The City and County Building in Salt Lake City, Utah, is one of these examples. The trend in voluntary earthquake hazard abatement is encouraging. Aside from the issue of life-safety, there is an increasing understanding that, from a business point of view, the cost of strengthening a building can be far less than its repair or replacement, espe-

cially if life-cycle costs are taken into consideration rather than initial construction costs.

Retrofit Methodologies & Techniques

Conventional Methods. There are a great number of methods and techniques that can be used to strengthen the seismic resistance of existing structures. The selection of the appropriate method is influenced by a great number of factors, such as:

- The structural system used in the original building construction;
- The architectural finishes (both exterior and interior);
- The internal function and use;
- The building configuration and shape; and,
- The building's historic significance.

The most common method is to add earthquake-resisting elements to the structure. These elements can be concrete shear walls, or moment frames or braces installed either externally or internally. A subgroup of these approaches is the addition of external buttresses.

Another method is to replace the existing components of the building with new lateral force resisting elements, such as replacing nonstructural core elements with new seismic resistant elements. The replacement or strengthening of existing structural components is also common. Existing masonry walls can be replaced with reinforced concrete walls, diagonal steel members can be added to steel frames to create braced frames, or wall openings may be closed in order to provide greater lateral strength. Figure 4 presents some of these methods.

Considerations in evaluating which seismic upgrading method to use are (see Figure 5):

The Capacity and Stiffness of the Floor Diaphragms. For instance, if shear walls are provided at the end walls of a rectangular building, diaphragm shears may exceed the capacity of the existing system forcing strengthening or replacement.

Existence or Capacity of Earthquake Force Collectors. Should the resisting elements be installed in the interior of the building, there must be a collector element that can transmit the inertial forces to the resisting elements.

Foundation Capacity. The new resisting elements will introduce additional loads on the footings, especially if the resisting elements (such as concrete shear walls or braced frames) are compact. High overturning forces, both compression or uplift, may require modifications to the foundations.

Structural Compatibility. It is crucial that the stiffness of new resisting elements is compatible with the flexural capacity of the existing structure. If the new element, such as a steel brace, will deflect under design load beyond the flexural capacity of the existing system, such as a masonry wall, the new element cannot protect the building from severe damage or failure.

Base Isolation. During recent years, the use of base isolation has become an accepted method in the seismic rehabilitation of existing buildings. The concept of base isolation is to uncouple the building from the ground by supporting it on isolating devices, making the building respond to the period of the isolator. As can be seen from Figure 6, a period shift from 1 to 2 or 2-½ seconds can reduce the building's response to the ground motion substantially. Clearly, the greater the period shift, the more effective the isolation systems become. However, should the building period coincide with that of the isolator, disastrous harmonic response will occur.

Base isolation is not a cure-all. Even though earthquake ground motion might be dramatically reduced, the reduced ground motion will still be transmitted to the structure. Other considerations — such as configuration, adjacency to other structures, overturning forces, etc. — must still be addressed. The motions of the building, sometimes as much as 18 inches, must be provided for. Utilities, elevators, stairs, etc., have to be designed and detailed to accommodate the motions of the building. Generally, base isolators have no tensile capacity; therefore, column uplift due to overturning forces must be avoided. Although the first cost of an isolator retrofit is generally higher than a conventional retrofit, many factors may make it a more desirable solution, especially true in the case of historic or landmark buildings where

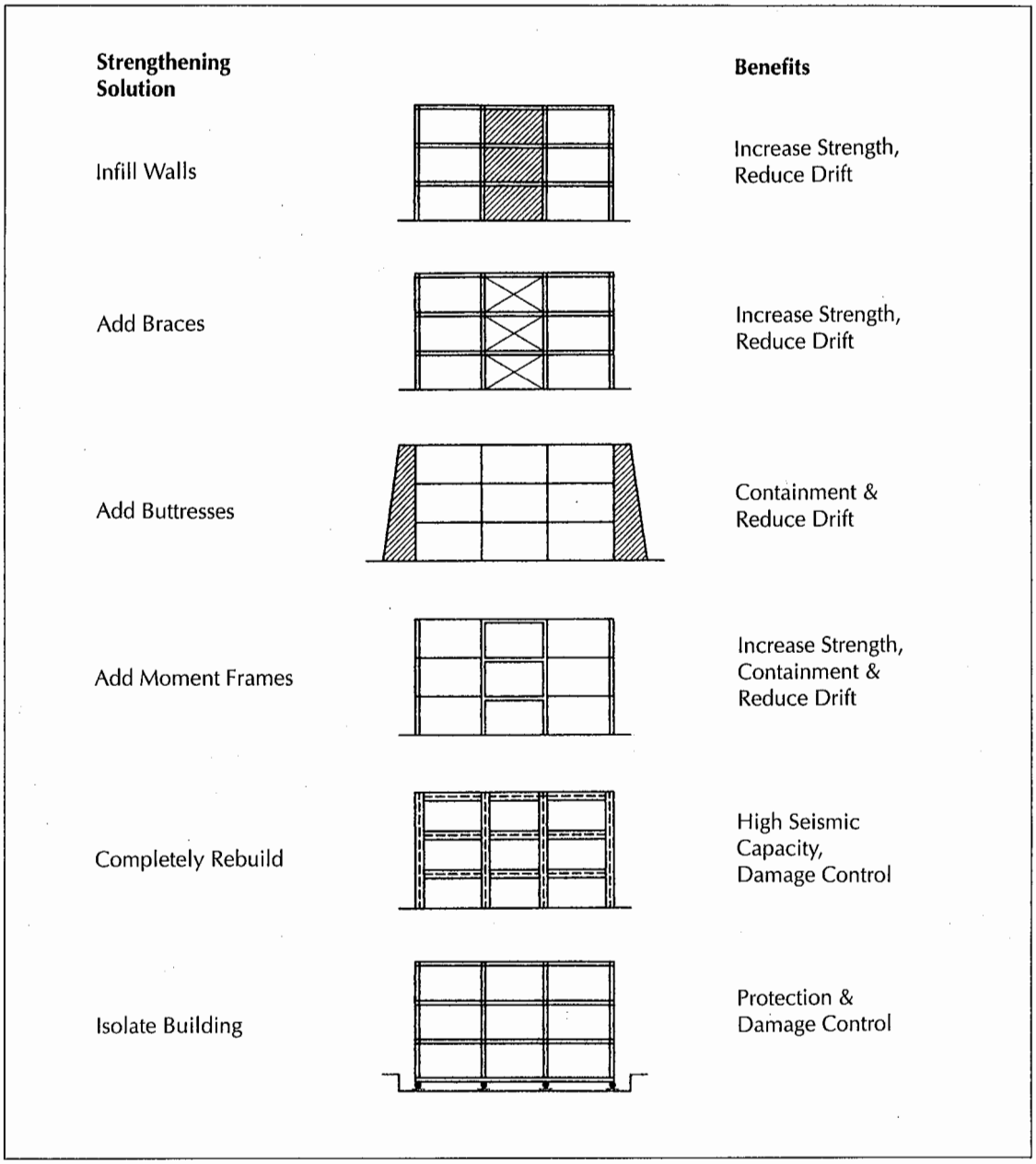

FIGURE 4. Conceptual solutions for the seismic upgrading of existing buildings.

the preservation of the historic fabric of the building is essential.

There are currently a number of isolating devices on the market and more in the development stage. Figure 7 shows three of the most common isolating devices: the high damping rubber isolator, the lead/rubber isolator and the inverted pendulum.

Passive Energy Dissipation. Traditional earthquake design practice permits the use of forces lower than those expected in the elastic design on the premise that, in a well designed and detailed structure, inelastic action will provide sufficient energy absorption to survive the earthquake without collapse. The object of energy dissipation is to introduce devices that are designed to dissipate energy and by doing so reduce the need of the primary structure to do

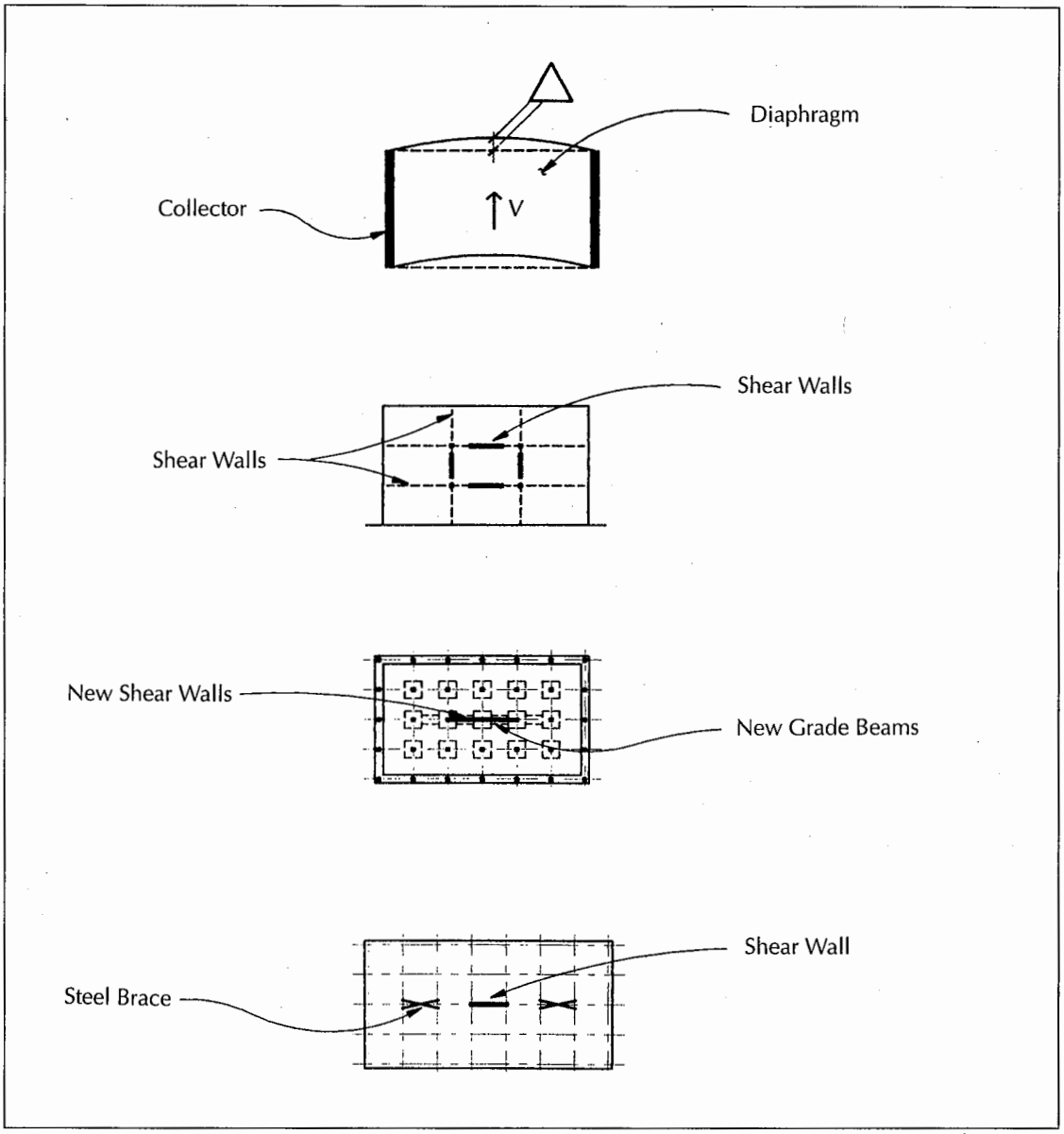

FIGURE 5. The application of different seismic upgrading systems on existing buildings.

so. In other words, energy dissipating devices add damping to a structure in such a way that they significantly reduce response to earthquakes. Some types of dissipation systems are visco-elastic dampers, friction dampers and lead extrusion systems.

Some examples where passive energy dissipation has been applied are:

- A one-story concrete frame building owned by Wells Fargo Bank in San Francisco. This building was damaged as the result of the Loma Prieta Earthquake in 1989. The building, approximately 80 feet square, was retrofitted by installing a steel frame at the four sides of the building. The brace was connected to the building with patented metallic-yield energy dissipators that provided added damping and stiffness.
- A number of buildings in Canada have been retrofitted using friction dampers.

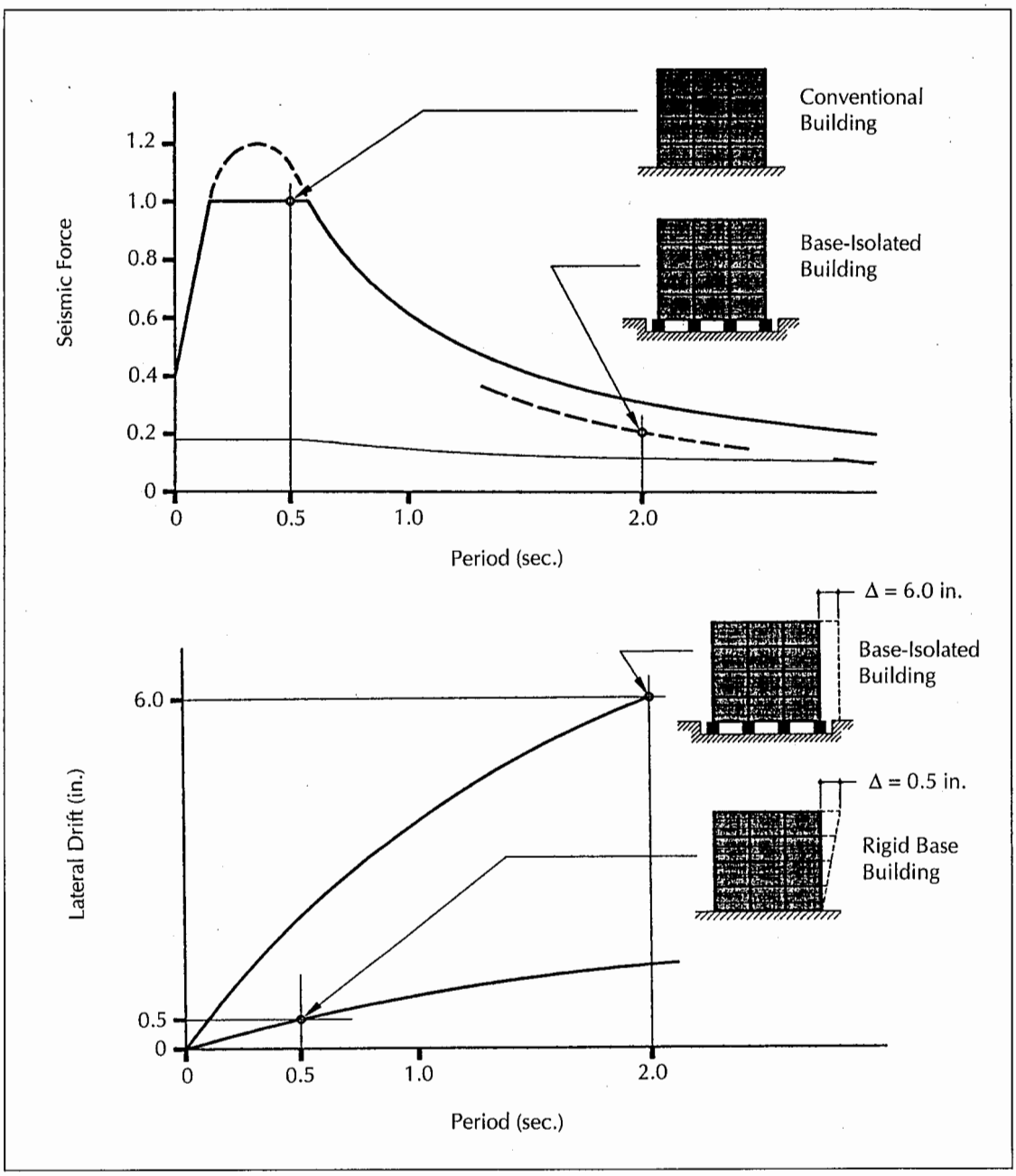

FIGURE 6. Base isolation response.

The devices consist of a series of specially treated steel plates that are clamped together and that are allowed to slip at a predetermined load.

More detail on these types of systems can be found in the proceedings of a seminar recently held by the Applied Technology Council (ATC-17.1).[5]

Active Energy Control. The development of active energy control systems is of fairly recent origin. Though there is much research being carried out in the United States, most of the work and applications have occurred so far in Japan. Generally speaking, the primary focus of active energy control has been on mass dampers.

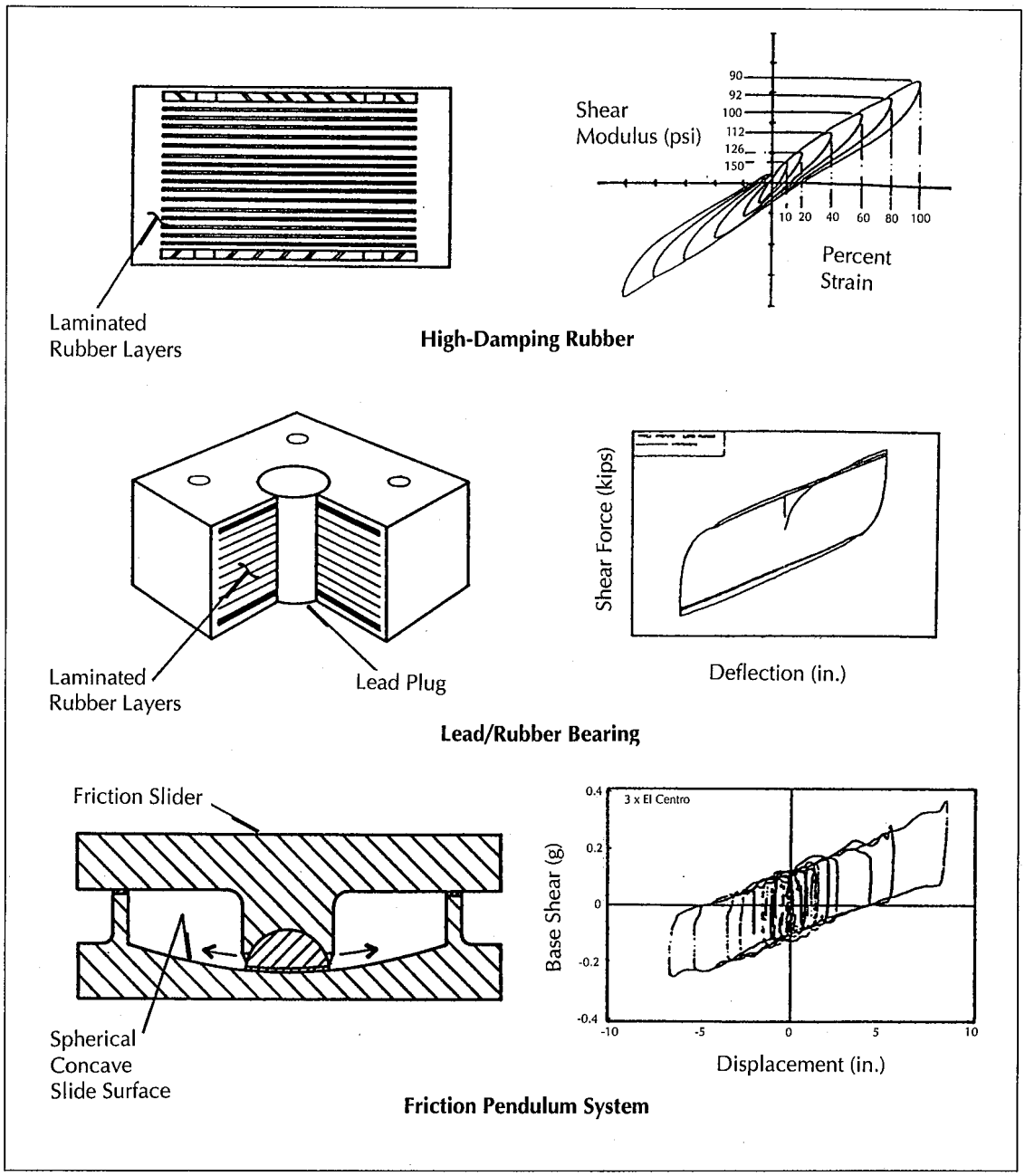

FIGURE 7. Isolation systems and hysteresis curves.

Importance of Case-by-Case Evaluation. The plethora of seismic retrofit methods and techniques should make it clear that there are not fixed solutions to the seismic strengthening of existing buildings. Each building represents a unique problem, the solution of which relies on careful study, ingenuity and experience. Some of the examples that follow have been chosen to demonstrate the great variety of problems and solutions involved in seismic retrofit.

IBM Product Development Center

In the early 1970s, the IBM Corporation in San Jose, California, embarked on a voluntary seismic hazard mitigation program. The motivation no doubt came from a greater awareness

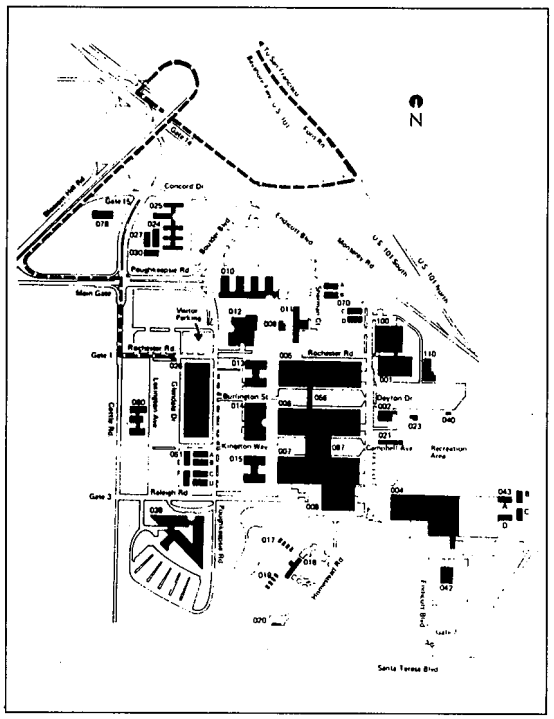

FIGURE 8. Site map of the IBM complex at San Jose.

of the risk of major earthquakes, possibly triggered by the NEHRP program, as well as a concern about life-safety and the post-earthquake operational capability of these important facilities. IBM retained a structural engineering firm to develop a hazard mitigation program. The work was carried out in stages because of the large number of buildings on the site (see Figure 8):

Stage I consisted of the rapid visual evaluation of all buildings. Drawings were reviewed and visual inspection performed. Evaluations were based on the building code used during design, and on the structural system used. Buildings were rated as either good, average or poor. Experience and judgment were major factors in these evaluations.

Stage II established priorities for further engineering studies, in coordination with facilities staff. Performance criteria were developed that set the acceptable design force.

In *Stage III* each building was analyzed for conformance with the design criteria, and a

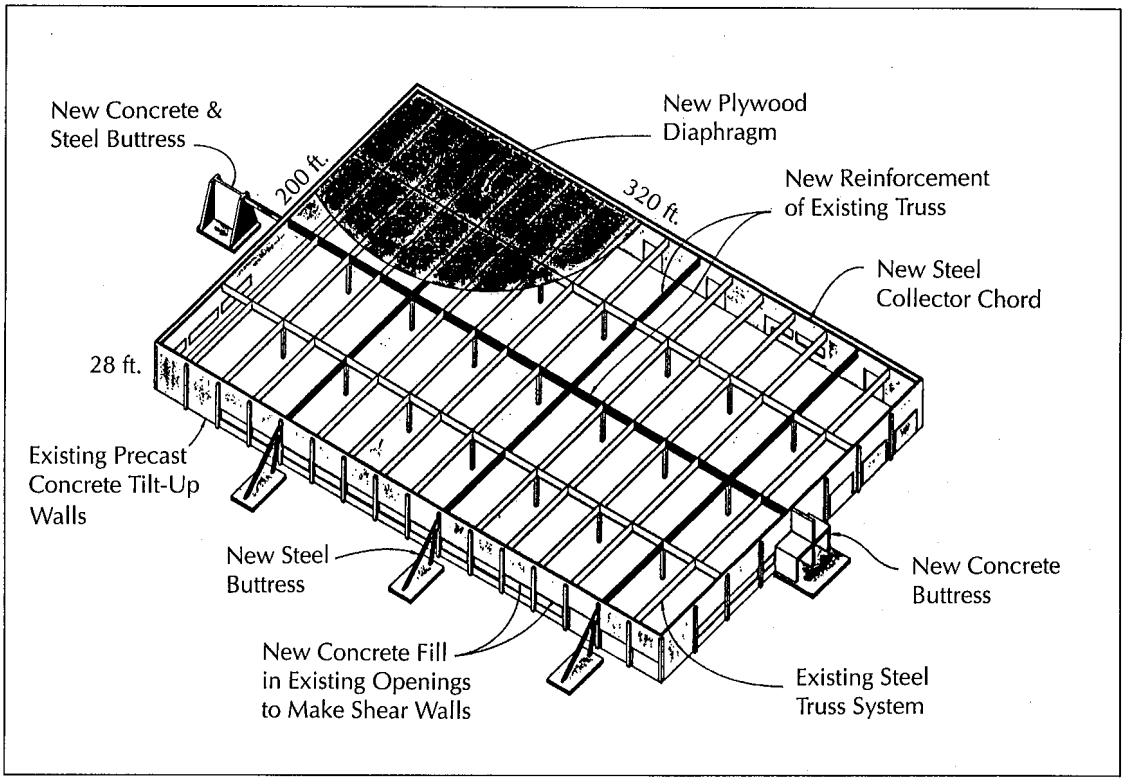

FIGURE 9. Schematic view of the seismic bracing for IBM Building 001.

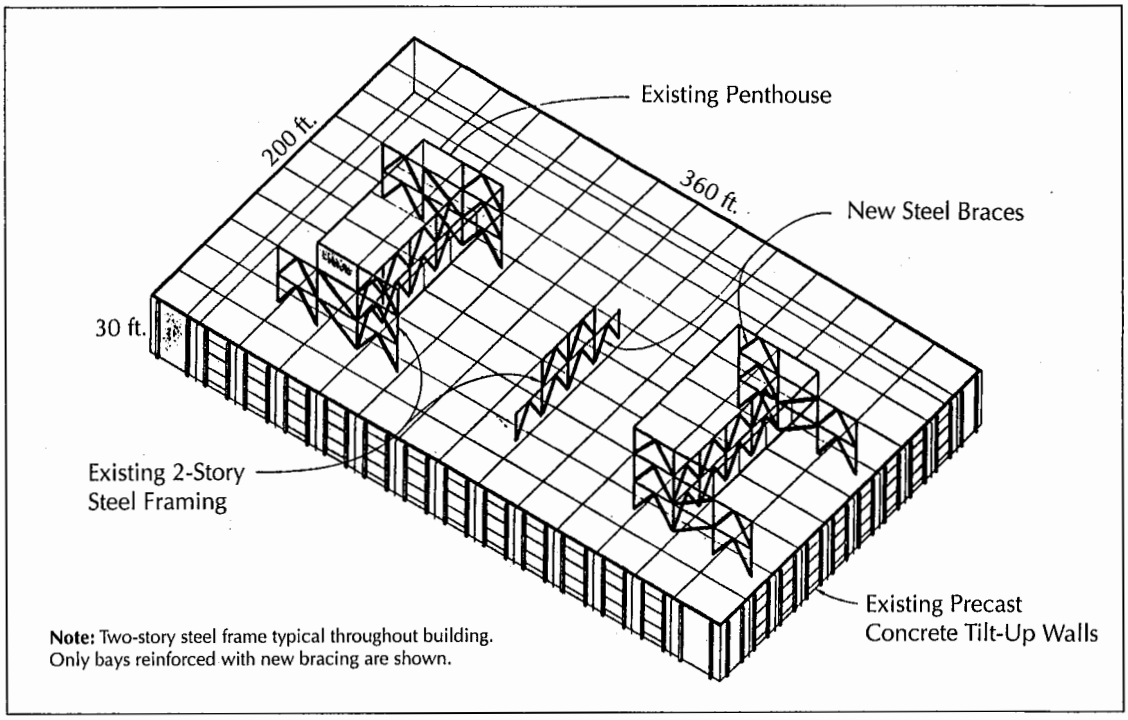

FIGURE 10. Schematic view of the seismic bracing for IBM Building 014.

retrofit concept was developed in accordance with the priorities established in Stage II (see Figures 9 and 10).

In *Stage IV* contract documents were prepared for the retrofit of each building, and bids were taken.

Stage V addressed the building source and process equipment. Criteria were developed for the seismic bracing of all ducts, pipes, mechanical and electrical equipment, and building contents. Because of the magnitude of this task, IBM's facilities management department retained a number of consultants to expedite this work.

IBM developed an in-house manual and trained staff to maintain the required level of earthquake preparedness. Subsequent earthquakes have proven that the work done is effective.

Unites States Geological Survey

The seismic retrofit of Buildings 1 and 2 at the United States Geological Survey (USGS) site in Menlo Park, California, is of note because of the constraints placed on the work. In the case of Building 1, it was to be kept fully occupied during the retrofit construction; in Building 2 (see Figure 11), only limited disruption was permitted. The buildings were upgraded to meet current UBC code requirements.

Building 1 is a light steel frame structure with a wood floor and roof. The adopted retrofit solution was to flank each side of the building with a moment-resisting exposed steel frame acting both in transverse and longitudi-

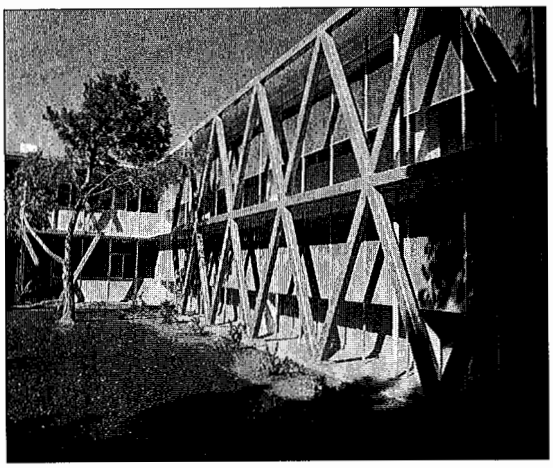

FIGURE 11. A view of USGS Building 2.

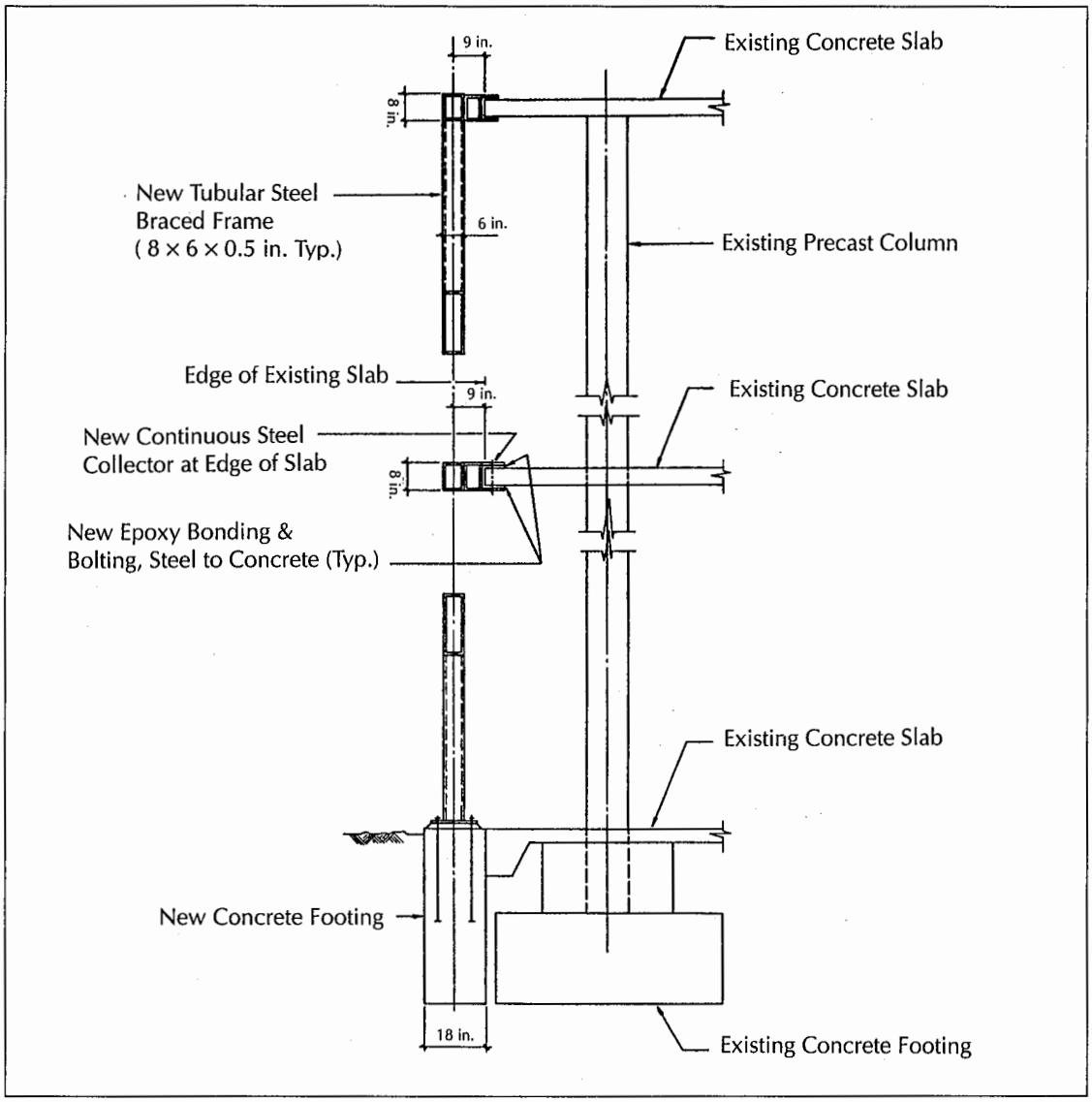

FIGURE 12. A section of the brace frame for USGS Building 2.

nal directions. The existing building was anchored to this new system. The exposed steel frame was designed to support a future horizontal expansion of this building.

Building 2 is a post-tensioned concrete lift slab structure. The major seismic strengthening elements are exposed tubular vertical trusses constructed with new foundations at the face of the building (see Figure 12). These trusses are connected to the existing concrete slabs that cantilever beyond the face of the building. Some new interior concrete shear walls were provided in sections closed off from the rest of the building (see Figures 13 and 14). Off-hour construction was used when needed.

Lawrence Livermore National Laboratories

Building 111, the Central Administration Building at the Lawrence Livermore National Laboratories (LLNL), in Livermore, California, was built in the late 1960s. The seven-story reinforced concrete structure was built in an unsymmetrical cross shaped plan, with a central core housing vertical circulation and utility shafts.

The building's vertical load carrying system

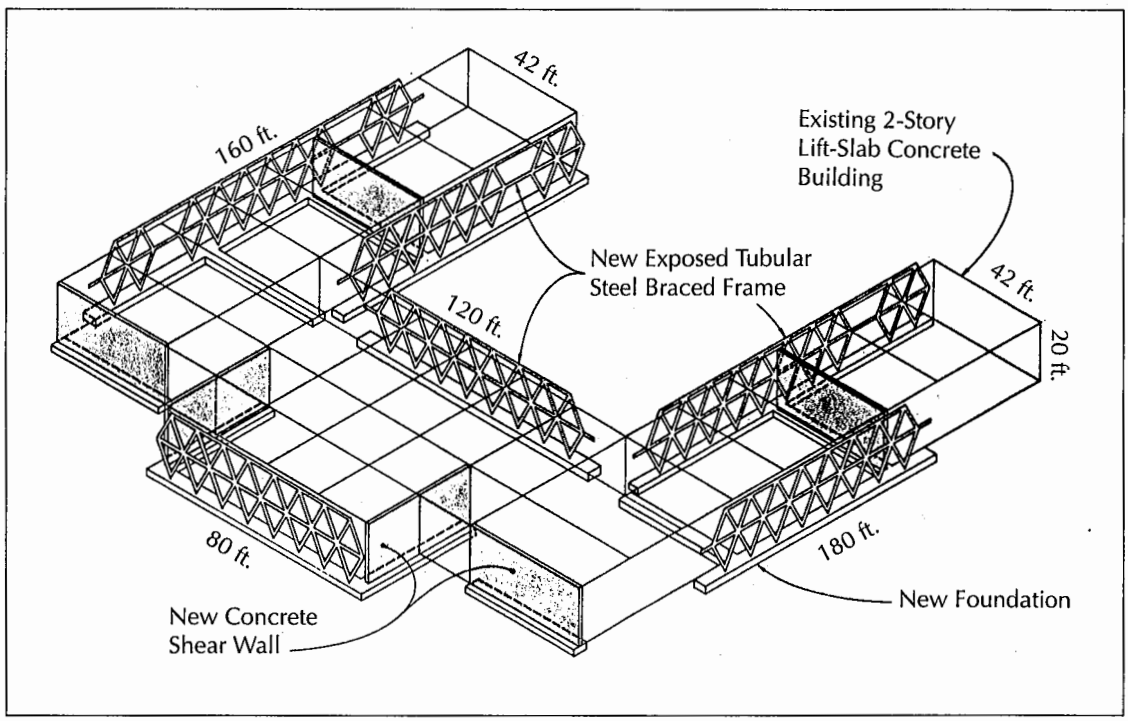

FIGURE 13. A schematic view of the seismic bracing for USGS Building 2.

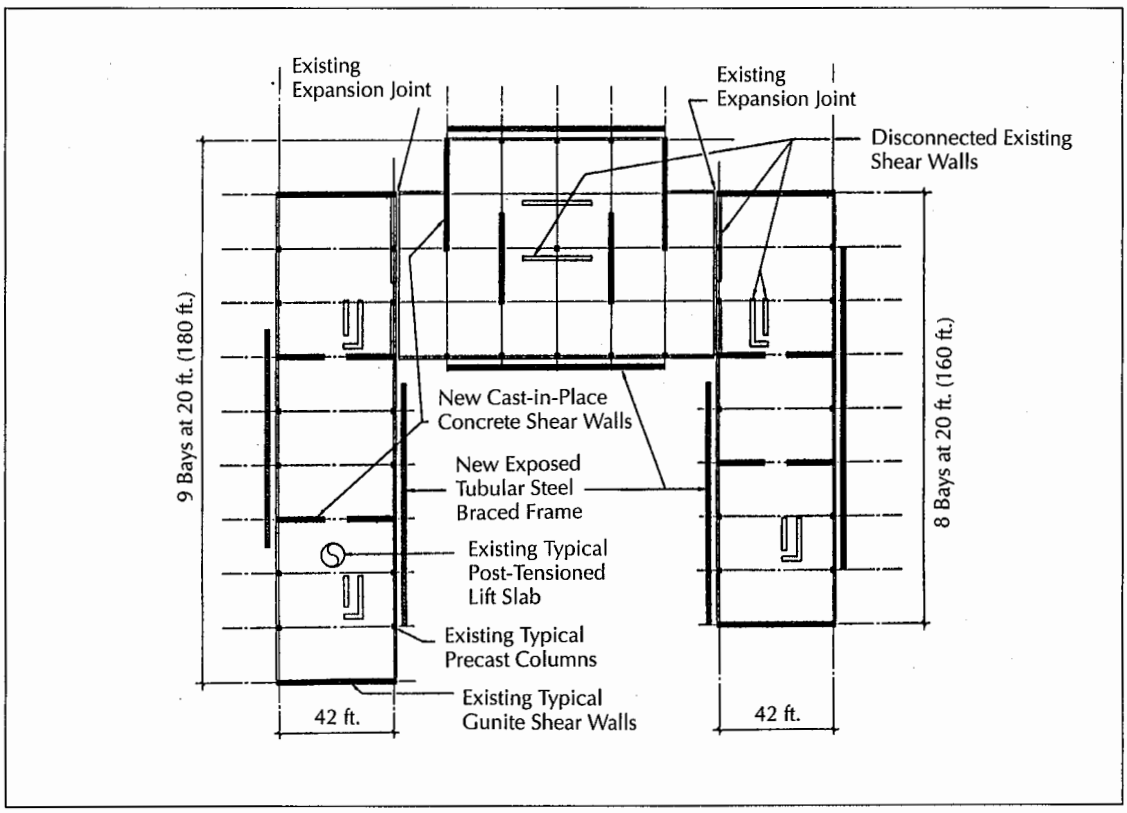

FIGURE 14. The seismic strengthening system for USGS Building 2.

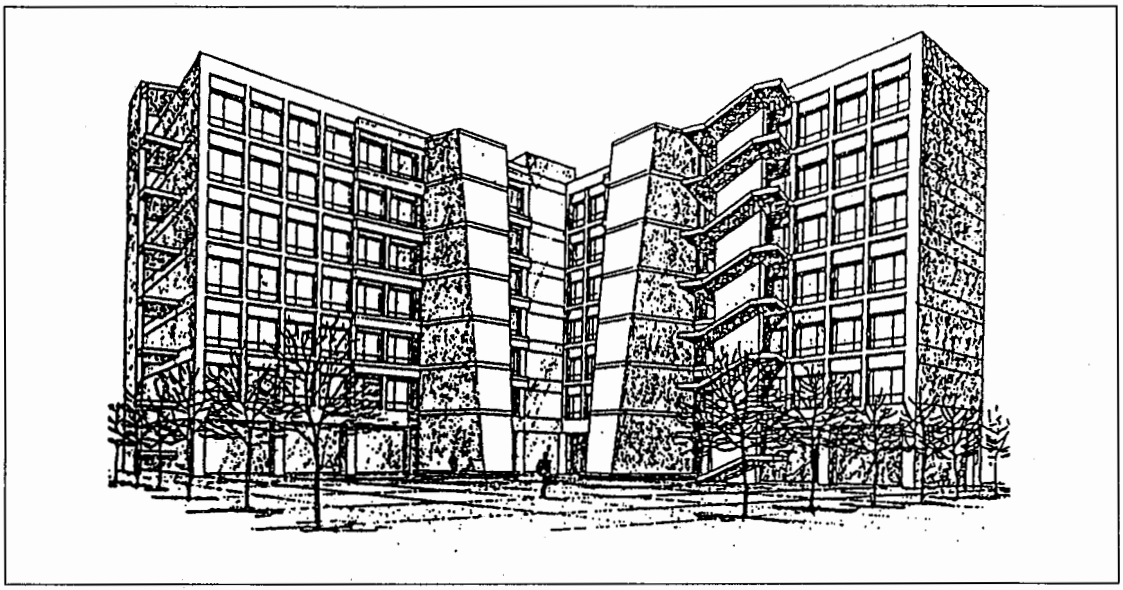

FIGURE 15. LLNL Building 111.

is composed of a 4-½ inch concrete slab supported on reinforced concrete beams spaced ten feet on center spanning across the wings where they are supported, in turn, by reinforced concrete columns. The existing lateral system is composed of reinforced concrete shear walls at the exterior ends of the wings, and shear walls enclosing the core.

The building was subjected to three earthquakes ranging in Richter magnitude from 5.1 to 5.8 in January 1980. Extensive cracking of shear walls, slabs and beam-column connections resulted. Repairs, accompanied by a limited degree of strengthening, were done after the earthquakes.

LLNL commissioned a full seismic upgrade in 1988. The seismic design criteria established for this building were:

- Non-collapse at a 0.5 g, ten percent damped, UBC shaped response spectrum, or UBC 1988 equivalent static approach (whichever is more stringent);
- Dynamic as well as static load analysis for comparison;
- Minimum disruption of occupants during strengthening; and,
- Minimum intrusion into the building during construction because of the sensitive nature of its contents.

The accepted solution to strengthen the building was to add two large tapered reinforced concrete box-section towers at the exterior of the two long building wings, with the

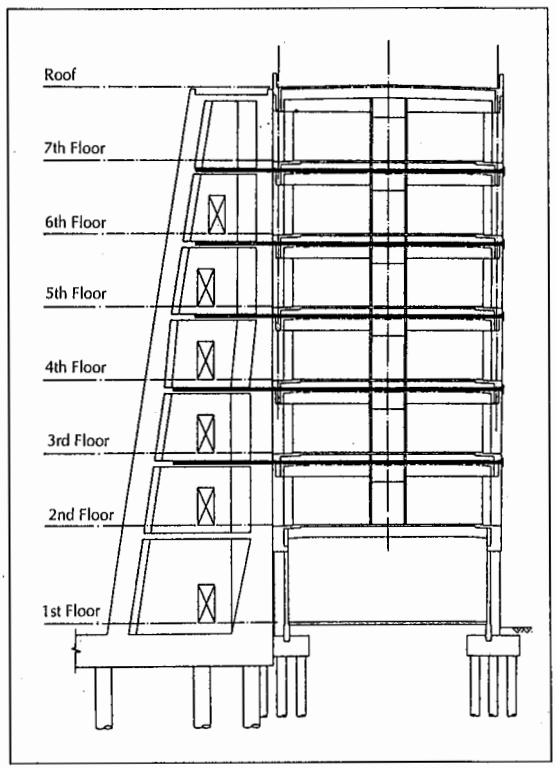

FIGURE 16. Section of LLNL Building 111.

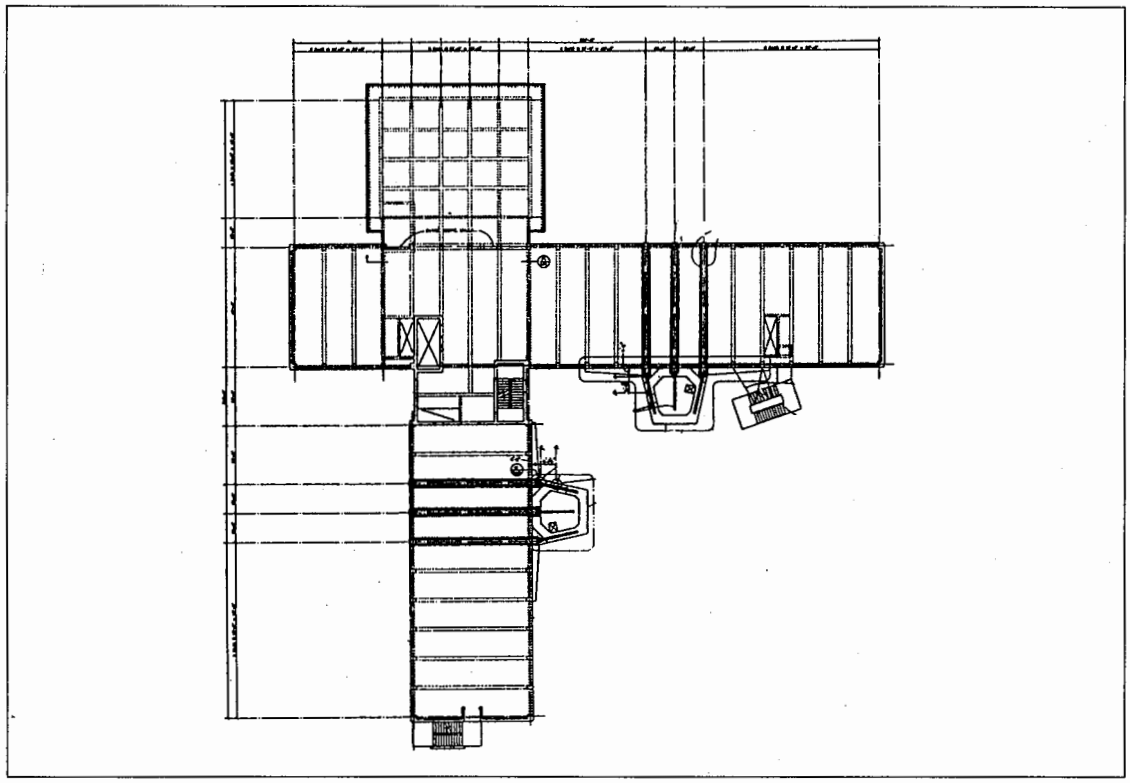

FIGURE 17. Plan view of LLNL Building 111.

towers located to minimize the stresses in the long diaphragms (see Figure 15). The towers rise from a seven-foot thick reinforced concrete foundation mat, which is supported on 38 three-foot diameter drilled reinforced concrete piers extending from 50 to 70 feet below the mat (see Figures 16 and 17).

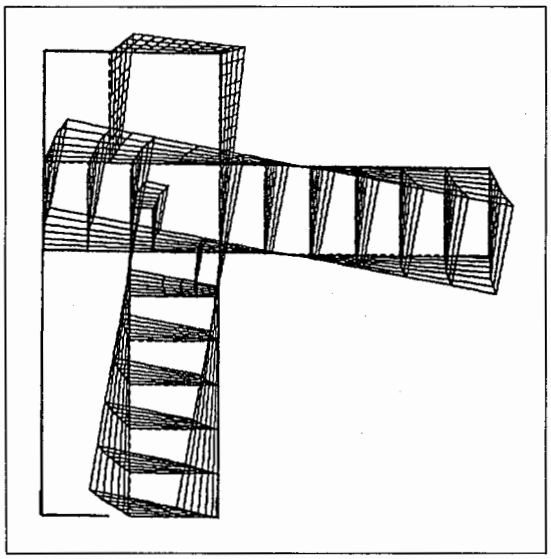

FIGURE 18. Three-dimensional model of LLNL Building 111.

The towers were connected to the building with channel-shaped steel collectors on each side of the floor beams. The collectors extend through the building terminating on a steel bearing plate anchor. The steel channels are epoxy bonded to the side of the reinforced concrete beams. Also, exterior reinforced concrete edge collectors were provided to transfer seismic shear to the towers, and interior 12-inch reinforced concrete shear walls were constructed in isolated locations to reduce the undesired torsional response of the structure.

Extensive analyses were made using four computer models (see Figure 18):

- A three-dimensional model was created (using a three-dimensional finite element program for structures with rigid diaphragms) that incorporated all new and existing structural components.

- A three-dimensional finite element program for modeling frames, shells and asolid elements was used. Foundation springs simulated the vertical and horizontal stiffness of the piers for this structural analysis program (SAP).
- A SAP computer model was also used to assess the effect of vertical accelerations on the structure.
- Lastly, a three-dimensional flexible diaphragm SAP model was developed to assess the effect of the flexibility of the diaphragms on the effectiveness of the newly added towers.

Three variations of the three-dimensional model for structures with rigid diaphragms were employed to simulate various building responses:

- A model excluding the interior collectors served to determine the compression force between the buildings and the tower;
- A model including the interior collectors determined the tensile forces in the collector; and,
- A full model, including the interior collectors and pile foundation, was fixed with a dummy level below the ground to simulate vertical, lateral and rotational stiffness of the pile foundation.

The overall construction cost for this project was below estimates, and construction was completed in 1991.

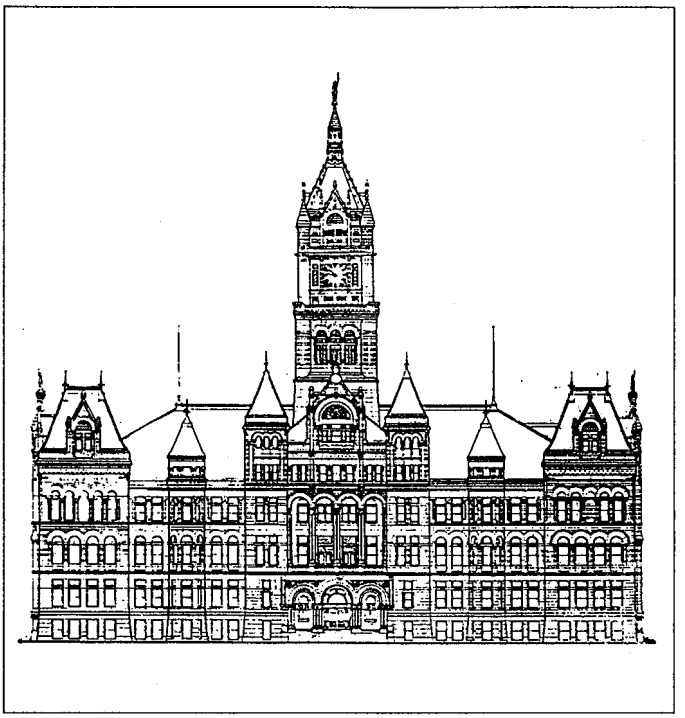

FIGURE 19. City and County Building, Salt Lake City.

City & County Building, Salt Lake City

The City and County Building in Salt Lake City, Utah, constructed in 1894, is a highly ornamented unreinforced brick and sandstone structure. It measures 130 by 270 feet in plan, with five main floors and a 12-story clock tower. This landmark building, with its 240-foot-high tower, dominates the surrounding landscape (see Figures 19 and 20).

The seismic vulnerability of the structure is aggravated by its proximity to the Wasatch Fault Zone. The building has a record of damage from various earthquakes, the largest of which occurred in 1934 with a Richter magnitude of about 6.1.

The building was constructed of unreinforced masonry and sandstone masonry walls with thicknesses of up to 24 inches for interior walls and 36 inches for exterior walls. Foundations are a combination of sandstone plinth and concrete footings. The first and fourth floors are framed with timber beams and planked with either wood floors or concrete topping. The second and third floors are framed with steel beams supporting shallow brick arches, which are covered with stone fill and a concrete topping. Anchorage between floors and walls is minimal. The roof framing is a steeply pitched timber system.

A number of alternative rehabilitation systems for the building were studied in the mid-1980s. The conventional systems envisioned utilizing concrete shear walls, which required the removal of costly architectural finishes such as oak wainscoting and ornamental plaster. In order to minimize the destruction and replacement of finishes, a base isolation system was selected for the retrofit.

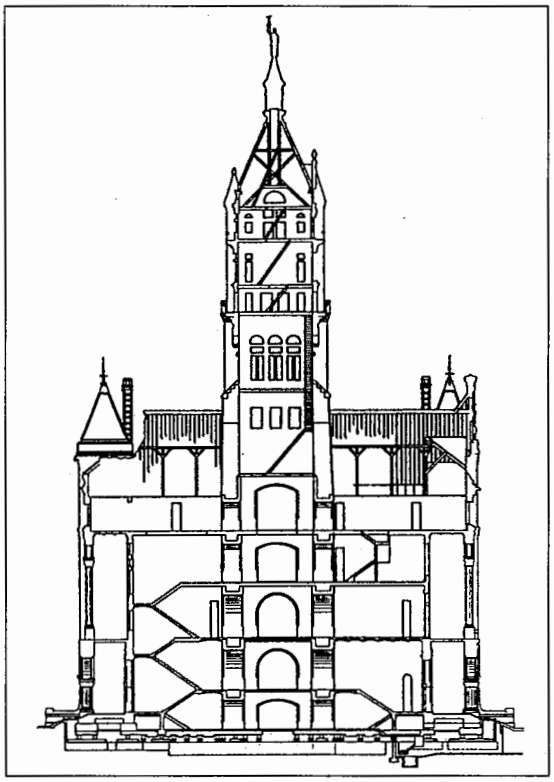

FIGURE 20. Section view of the City and County Building, Salt Lake City.

A site-specific response spectra indicated a free-field ground acceleration of 0.20 g, with a site period of 1.0 to 1.4 seconds. A base-isolation system with an effective fundamental period of 2.0 seconds reduced the base design shear to 0.07 g.

The base-isolation solution required the replacement of the first floor with a concrete slab on a metal deck supported by steel beams. Partial replacement of other floor diaphragms was required to provide continuity. A plywood diaphragm was installed at the ceiling level of the fourth floor, in lieu of a roof diaphragm, to support the tops of the masonry walls. Below the first floor, the masonry walls were "clinched" between pairs of reinforced concrete side beams, tied together by regular cross beams and ducted post-tension rods.

Once the concrete side beams were installed, portions of the masonry walls and plinth under the cross beams were removed, permitting the installation of the base isolators (see Figure 21). Four hundred and forty-seven isolation bearings were placed under the building. The isolators were preloaded to minimize axial shortening of the bearings. After installation, the

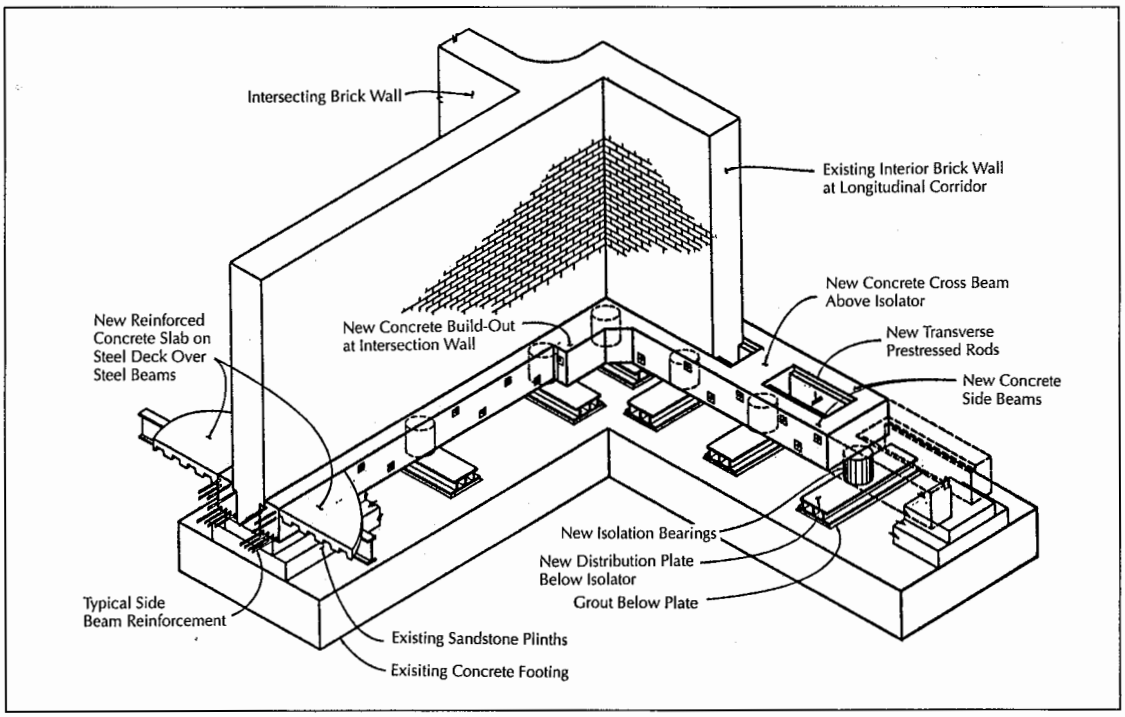

FIGURE 21. An isometric cutaway at an interior wall intersection (Salt Lake City).

balances of masonry walls and plinth stone were removed, permitting the isolators to translate freely.

The computer model for this project combined a linear elastic superstructure with the non-linear base-isolation elements. Shell elements were used to represent the interior masonry walls, and beam and column elements were used to represent the highly fenestrated exterior walls. Diaphragms were modeled using elastic properties that corresponded to the existing or proposed concrete toppings. Information on existing material properties was based on in-plane shear tests and prism test results.

The computer analysis consisted of a response spectrum analysis using a non-linear spectrum to account for the behavior of the bearings. This analysis yielded approximate internal forces and displacements for use in wall and diaphragm checks. The output of this analysis was the basis for the design of the structural elements.

To validate the response spectra analysis and the design, a non-linear time history analysis was done using the complete elastic superstructure model on top of the non-linear bearing elements.

The base isolators were bid on the basis of a performance specification. The low bid provided lead rubber bearings. The seismic rehabilitation was completed in 1989. Cost of the project was $6 million for the structural work, $30 million overall.

Oakland City Hall

The Oakland City Hall is an 18-story structure with full basement. Completed in 1914, it is the first high-rise governmental building in the United States. The structure is steel frame with unreinforced masonry infill. Cladding and ornamentations are terra cotta. A ten-story office tower rises above a three-story podium that houses a central rotunda, council chambers, and administrative offices of the mayor and city manager (see Figures 22 and 23). Above the office tower a two-story transfer system supports a 91-foot-high clock tower. The entire structure rises 324 feet above the street. The building is on the National Register of Historic Places.

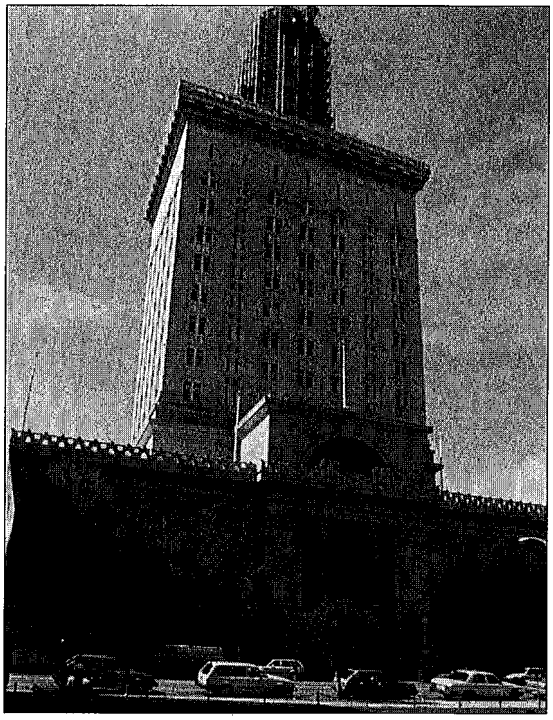

FIGURE 22. Photograph of the Oakland City Hall.

The building suffered extensive damage during the Loma Prieta Earthquake in 1989 and was declared unsafe for occupancy. After numerous studies, which were reviewed by federal and local agencies, it was decided that the building could be rehabilitated using base isolation.

The need to preserve the historic fabric of the building made the direct application of the UBC unfeasible. Rather, a "performance-based" approach was developed, which would protect life-safety during large earthquakes. The design of the superstructure was based on a site-specific response spectrum with an earthquake-return period of 475 years (the same as required by the UBC). The maximum credible earthquake, used to check the stability of the isolation system, corresponded to a magnitude 7.0 event on the nearby Hayward Fault.

The anticipated performance of the retrofitted building permits some minor yielding in the new steel bracing of the office tower. However, any damage caused by such low magnitude movement would not require repairs; and masonry elements would be expected to expe-

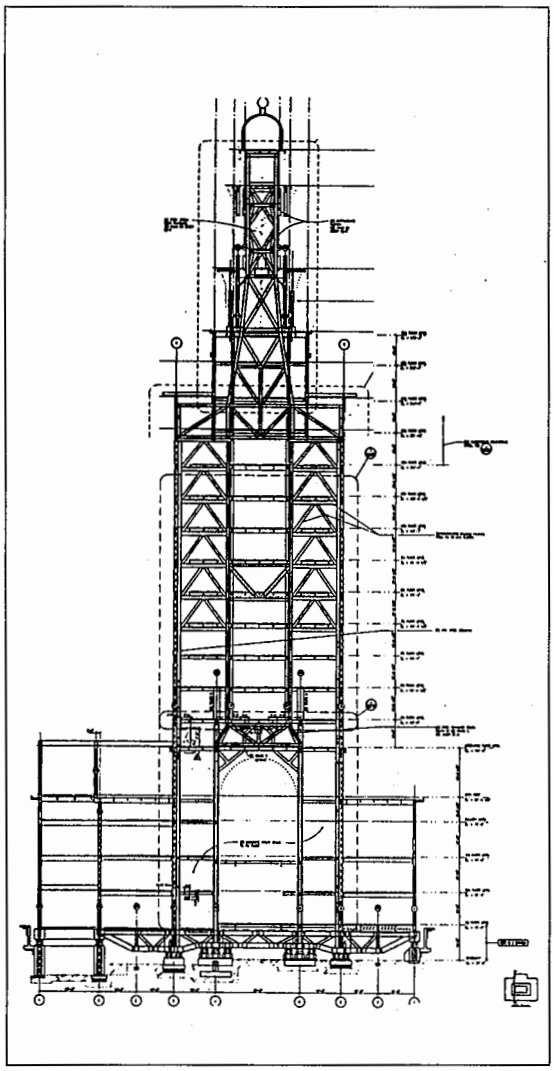

FIGURE 23. Section view of the Oakland City Hall.

rience minor cracks, which would be repairable.

The seismic upgrade called for a braced steel frame in the clock tower, which would resist the entire tributary load without dependence on the existing masonry. Interstory drift of the clocktower then would be limited to 0.008 times story height for the design earthquake.

The clocktower braced frames are supported on a series of one-story deep transfer trusses spanning 62 feet to new steel columns that extend to the basement of the structure. A new diagonally braced horizontal diaphragm transmits clocktower seismic shear forces to the perimeter walls of the office tower. The existing office tower exterior masonry walls are used to resist lateral loads in combination with new vertical steel braced frames, where needed. Stiffness compatibility between the two systems was determined by extensive in-situ dynamic testing and finite element analyses to determine the strength and stiffness of the masonry elements. Ultimate shear strains of the unreinforced masonry walls were limited to 0.001 in/in.

Below the seventh level of the office tower and podium, new concrete shear walls were introduced to resist the lateral forces. The concrete shear walls terminate on new 8-½ foot deep steel "outrigger" trusses in the basement story, which distribute the overturning forces over a broad footprint (see Figure 24). Horizontal steel braces form a diaphragm below the first floor to deliver the lateral loads to a system of 111 lead-rubber isolation bearings. These iso-

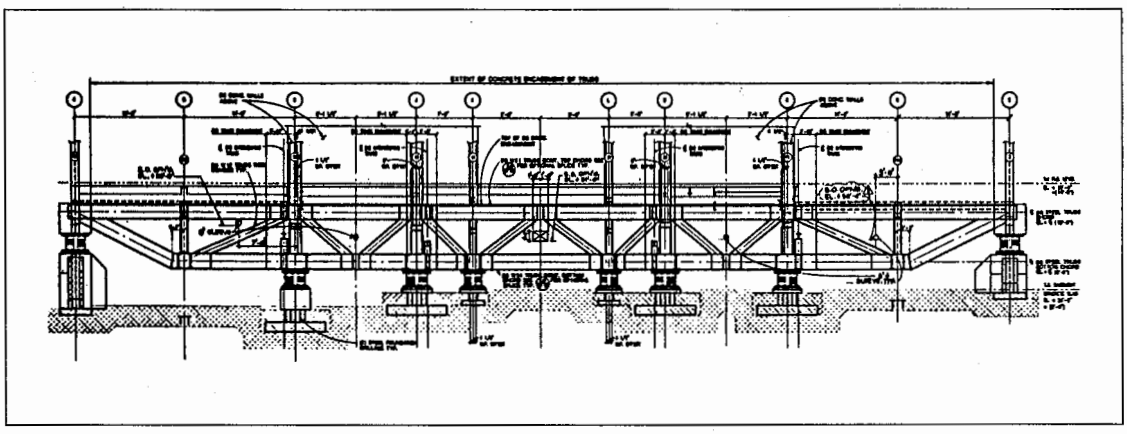

FIGURE 24. Truss section at the Oakland City Hall.

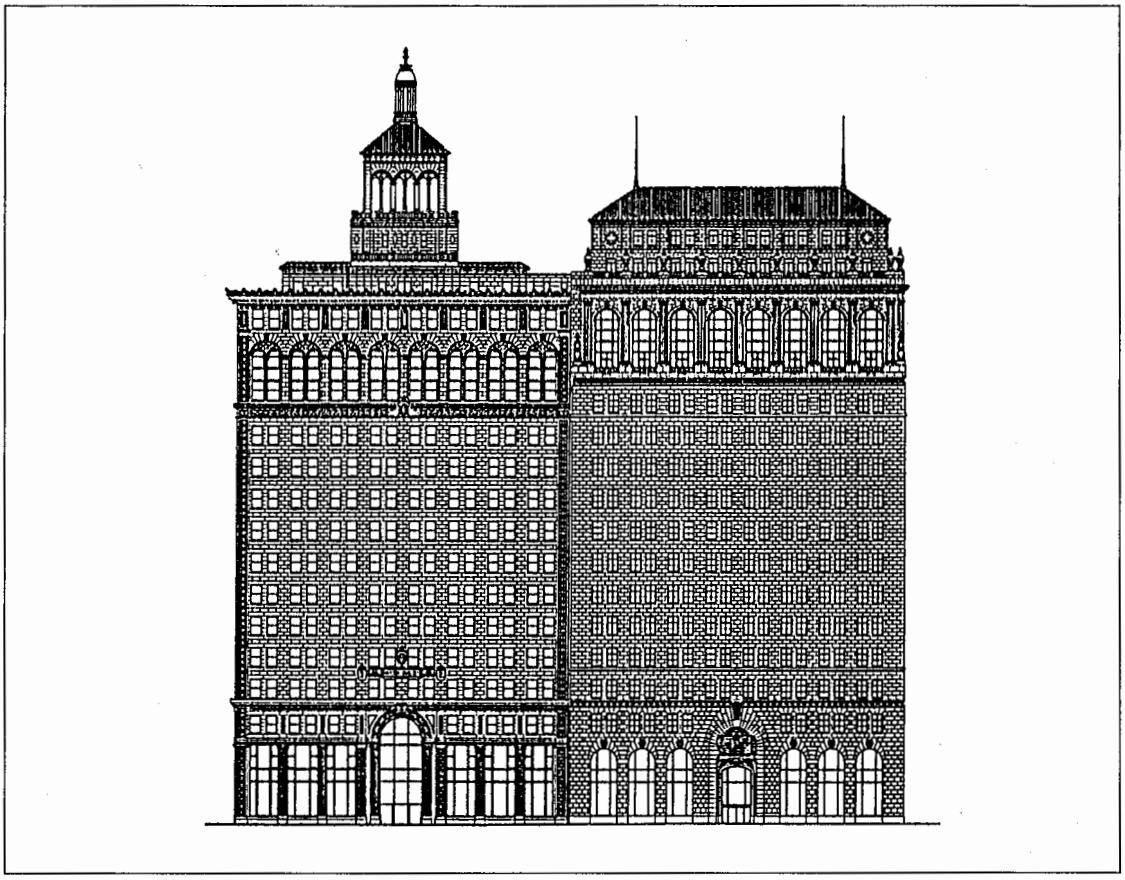

FIGURE 25. North elevation of the PG&E Market Street complex.

lators rest on steel/concrete pedestals that are supported on the existing mat foundation. A gap around the building permits the building to move 17 inches during a magnitude 7.0 event on the Hayward Fault.

In order to assess the seismic response of the building, a series of dynamic modal analyses were performed. In order to verify the results of the modal response analyses, and to assess the effect of the isolator hysteretic behavior, non-linear time history analyses were done. The time history analyses utilized three sets of ground acceleration records that were modified to match the site-specific design spectrum.

Historic hollow clay tile interior finishes are being repaired through a combination of self-tapping anchors, metal lath, epoxy and cement plaster. Damaged exterior terra cotta cladding elements will be replaced using new pieces cast in molds formed from salvaged damaged pieces.

The construction is currently well under way, with completion scheduled for late 1994. The total construction cost is estimated to be $76 million, with structural rehabilitation costs of $19.7 million. The cost of the isolation bearings is $2 million.

Pacific Gas and Electric Company

The Pacific Gas and Electric Company (PG&E) complex occupies one block in the financial district of San Francisco. Located at 215/245 Market Street, the seven-building complex houses the company's corporate headquarters and the company's critical energy management system (see Figure 25).

The October 1989 Loma Prieta Earthquake caused significant disruption and some damage to the buildings within the Market Street complex. The bulk of damage was concentrated in the brittle terra cotta facade of the buildings.

In March 1992, PG&E commissioned the seismic retrofit of four of the seven buildings in the complex. This project included two 17-story buildings built in the 1920s, and one seven-story building and one 14-story building built in the 1940s. The seismic retrofit program calls for complete reconstruction, upgrading the buildings to modern Class A high-rise office standards.

The two 17-story office buildings are historic landmarks. The structures of each are similar: structural steel frames, concrete slabs and unreinforced masonry infill — all bearing on a timber pile foundation system. The masonry infill is finished with glazed terra cotta stone. The two buildings are "L" shaped in plan and form a larger "U" due to their adjacency.

The 1940 annex buildings were constructed using structural steel frames and concrete floor systems with reinforced concrete infill walls as opposed to the unreinforced masonry of their predecessors. The 14-story PG&E annex was designed to resist earthquake forces using concrete wall elements in conjunction with the steel frame. The building supports a terra cotta finish identical to the original high-rise.

PG&E developed seismic performance goals greater than current code standards for the proposed seismic retrofit because this complex houses critical energy management systems. These goals express the owner's expectations for seismic performance after a major earthquake.

The goals that were developed to minimize risk to life were:

- Maintain building stability;
- Minimize falling hazards; and,
- Ensure survival of life-safety systems, stairwells, and egress routes.

The goals to minimize disruption of normal services and operations were:

- Ensure immediate access to critical facilities;
- Maximum of four weeks to restore full operation; and,
- Anticipate an extended period of non-disruptive repair of architectural and minor electrical/mechanical items.

Other criteria and physical constraints affecting the retrofit were:

- Maintain the existing historic character and details;
- Maintain the energy management system facility without interruption;
- Soft soil site;
- Limited separation between adjacent buildings;
- Limited capacity of existing foundations;
- Limited capacity of existing steel frame; and,
- Brittle terra cotta facades.

Reinforced concrete was selected as the material for strengthening the existing steel frame. It was chosen primarily for its ability to achieve a high lateral stiffness that is compatible with the existing masonry and terra cotta materials. The structural system is a dual frame-wall system that is detailed to deform in a ductile manner.

The frame-wall system is located in plan around the inner court walls of the complex and forms a high-rise "U" configuration (see Figures 26, 27 and 28). This configuration provides optimal distribution of overturning forces from the two interconnected buildings to their respective foundations. Additional wall-frames are located at the ends and mid-portion of the main U shape, serving primarily to further distribute overturning forces to the foundation. The location of these new walls was determined by where they would have the least impact on the historic features of the buildings. The inner court walls were not clad with the historically sensitive terra cotta system used for the street side facades.

The re-entrant corners of the wall-frame system include a rectangular core around existing elevator shafts attached to a triangular "pylon" element. The purpose of the pylon is to provide a load path for diaphragm forces at the re-entrant corners of the building without crossing the existing elevator shafts.

The entire wall-frame system is supported along its base by a three- to five-story-high wall-beam that stiffens the lower stories, thus eliminating the soft story configuration present in the existing building.

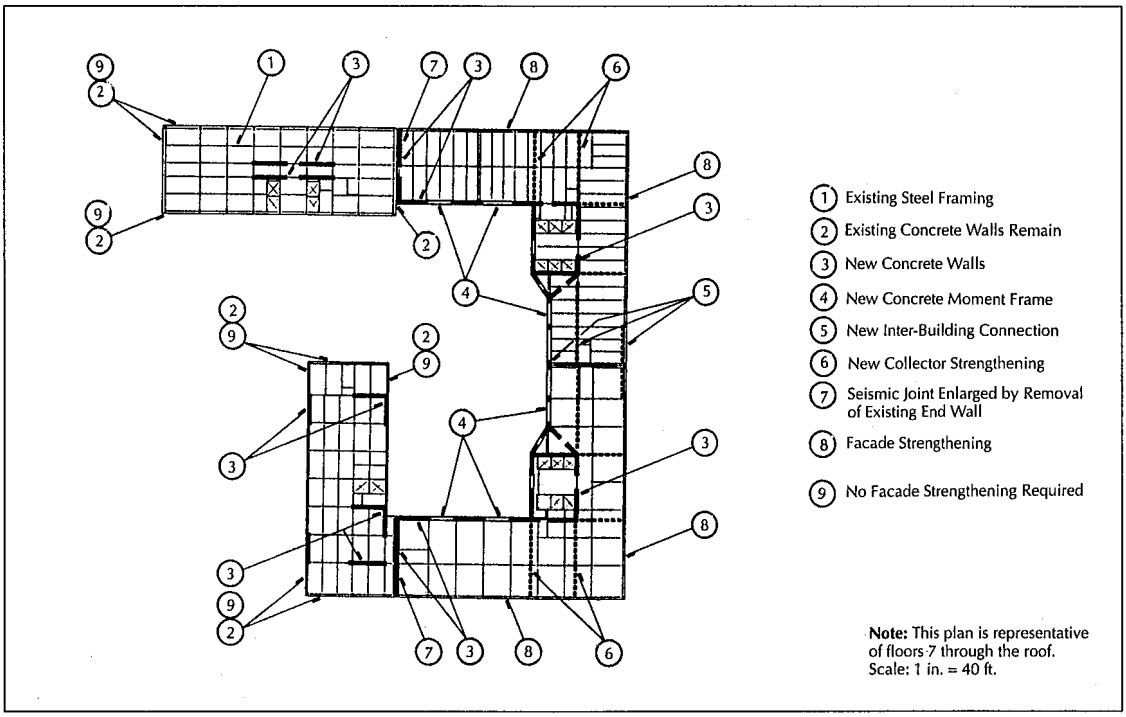

FIGURE 26. Upper level framing plan for the PG&E Market Street complex.

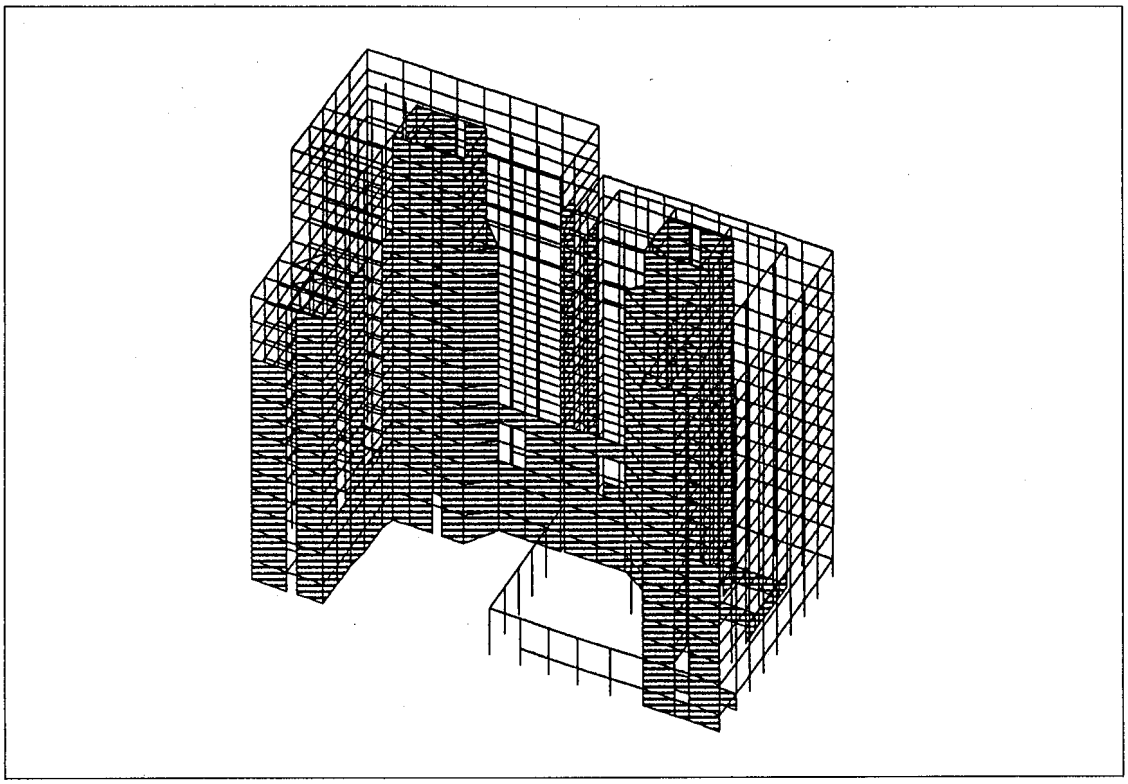

FIGURE 27. A view of the seismic retrofitted PG&E Market Street complex, looking towards the northwest.

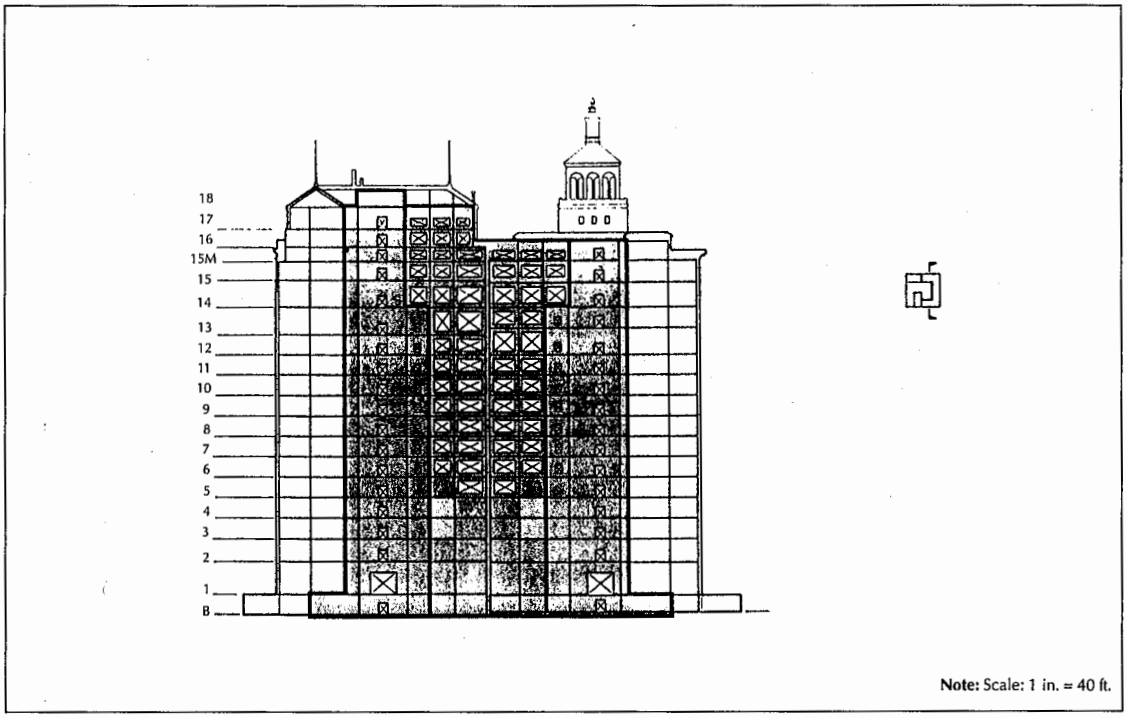

FIGURE 28. South court wall elevation of the PG&E Market Street complex.

The existing floor diaphragms were strengthened by introducing "collector" elements that tie the extreme ends of the building to the new wall-frame system. Separations between the buildings were enlarged by various means in order to minimize the possibility of pounding.

Analysis showed that the masonry/terra cotta piers would begin cracking at drift ratios of about 0.001, whereas a major earthquake would cause drift ratios of between 0.006 and 0.01. A solution that consists of mounting new terra cotta replicas of the originals to a precast concrete panel would have resulted in a loss of historic status for the buildings and, therefore, was not acceptable.

The method devised for isolating the masonry-terra cotta piers from large earthquake deformations was to reinforce the masonry from the inside of the building with a light reinforced concrete skin, and "articulating" the facade by cutting a thin horizontal joint completely through the terra cotta, masonry and concrete at the top and bottom of each pier. The articulation joints allow interstory deformations without inducing the large diagonal tension and compression forces into the masonry piers (see Figures 29 and 30).

An analysis program was established as follows:

- Develop three-dimensional finite element models of each building — modeling flexible foundations, flexible diaphragms, existing structural elements and new structural elements;
- Perform parametric dynamic response spectrum analysis of each building — using unreduced maximum probable and maximum credible site-specific earthquake spectra, and studying the influence of assumptions such as pile stiffness and cracked section properties on forces and displacements;
- Using the detailed three-dimensional finite element models as well as a linear elastic based computer program, perform an approximate non-linear incremental static push-over analysis to collapse (the "capacity-spectrum" approach was used to determine the peak displacements); and,

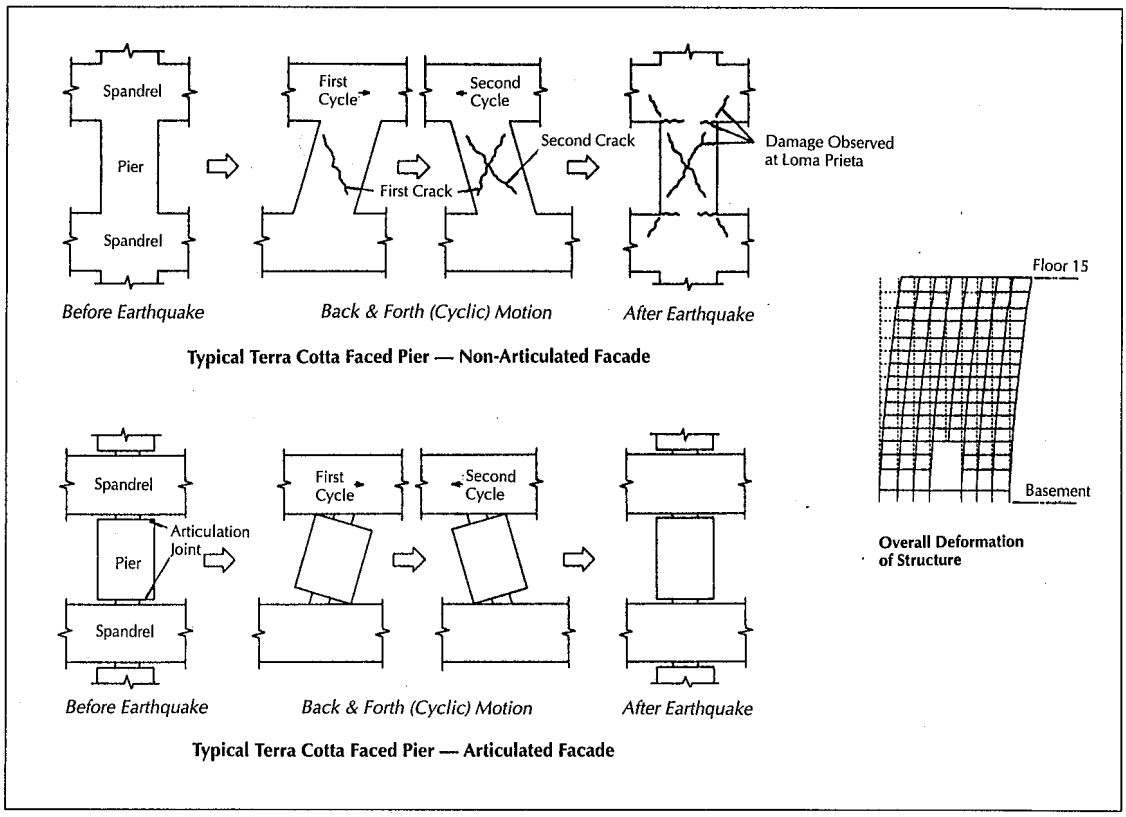

FIGURE 29. Seismic response of the facade for the PG&E Market Street complex.

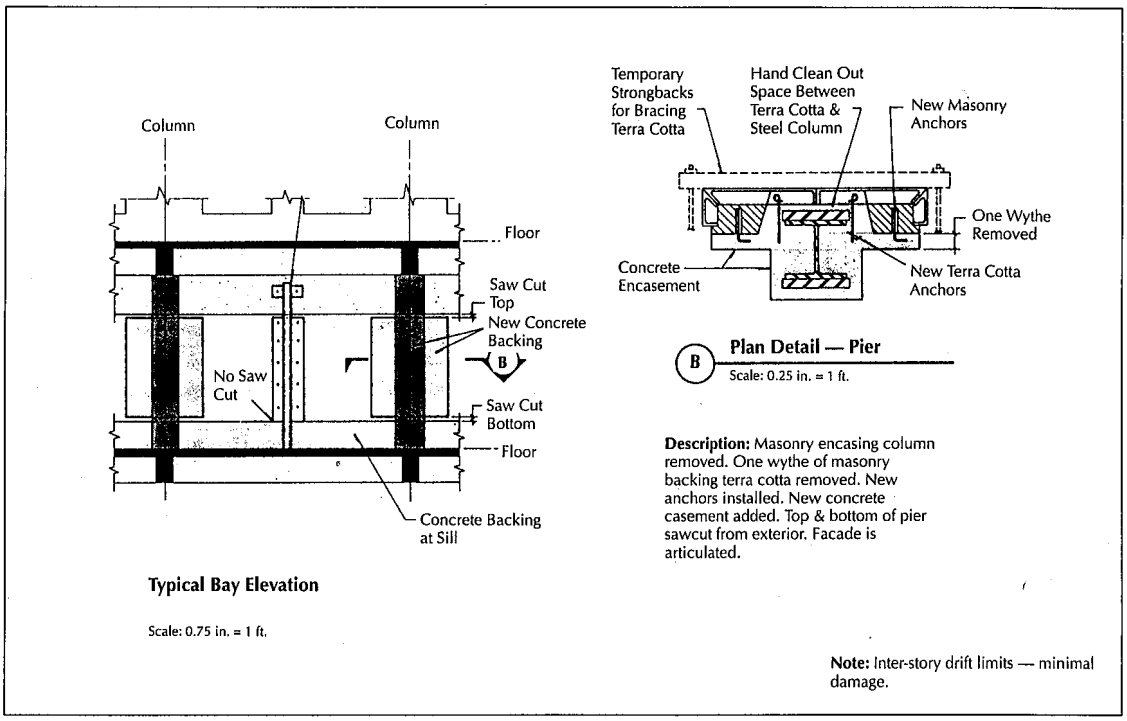

FIGURE 30. Articulated facade details for the PG&E Market Street complex.

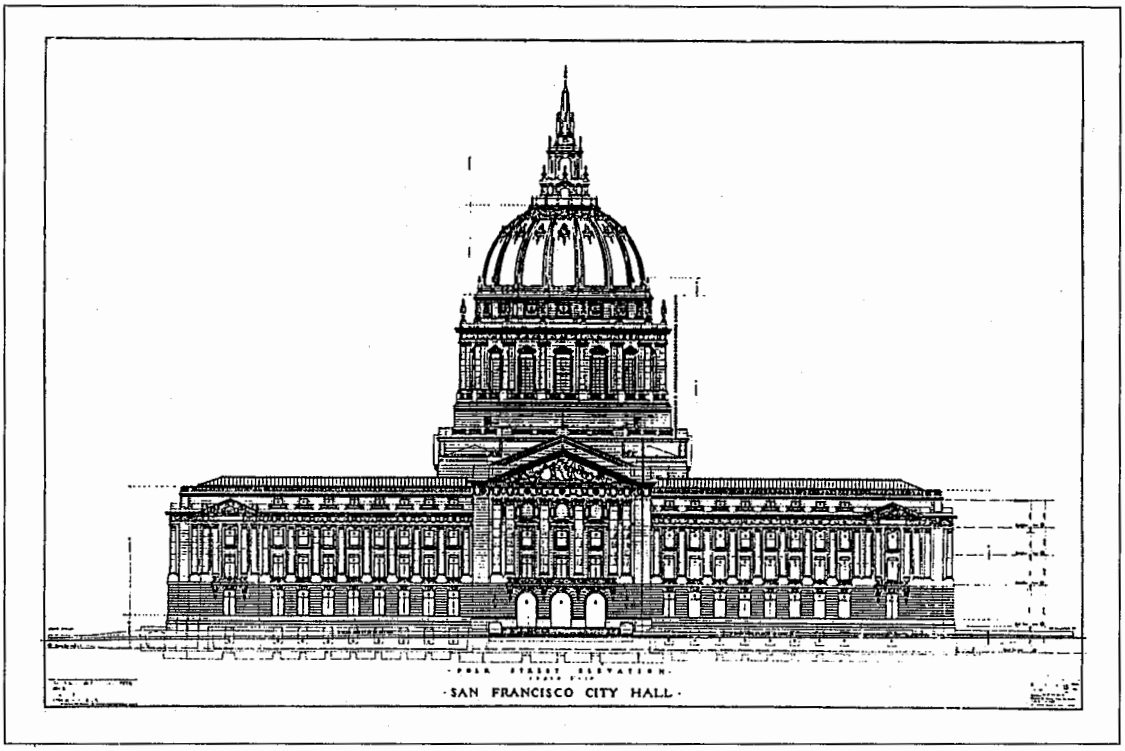

FIGURE 31. Elevation of San Francisco City Hall.

- Determine moment-curvature relations for individual members so that post-elastic deformations could be equated to compressive and tensile strains in the concrete/steel components (this analysis confirmed that the structure could undergo the maximum projected displacement with limited structural damage).

The seismic retrofit of the PG&E Market Street complex is a special case of seismic hazard mitigation. The owner's seismic performance goals surpassed to a significant degree what is normally expected of a structure exposed to a major earthquake. The use of approximate non-linear analysis techniques on structures of this scale is appropriate given the nature of the owner's expectations.

San Francisco City Hall

The San Francisco City Hall is located in the San Francisco Civic Center National Historic Landmark District. Designed in 1913, it replaced the city hall building that was destroyed by the 1906 San Francisco Earthquake.

The five-story building with its rectangular plan occupies approximately two city blocks with dimensions of 309 by 408 feet (see Figures 31 and 32). Its landmark dome rises approximately 300 feet above a central circulation area known as the rotunda. The main entrances to the building open into the rotunda area, which is ornately finished by marble, limestone and cast plaster. The exterior of the building is clad with a facade of ornately detailed granite from the foothills of the Sierra Mountains.

The building structure is composed of a complete steel frame with reinforced concrete slabs. The exterior granite facing is backed with unreinforced brick masonry that was laid-up integrally with the granite. Many of the infill partition walls were constructed with hollow clay tiles covered with cement plaster. The dome is a multi-tiered steel structure supported on four steel column towers located at corners of the rotunda. The exterior dome trusses were infilled with hollow clay tiles covered with lead and copper roofing.

The building was damaged during the 1989 Loma Prieta Earthquake. The damage was con-

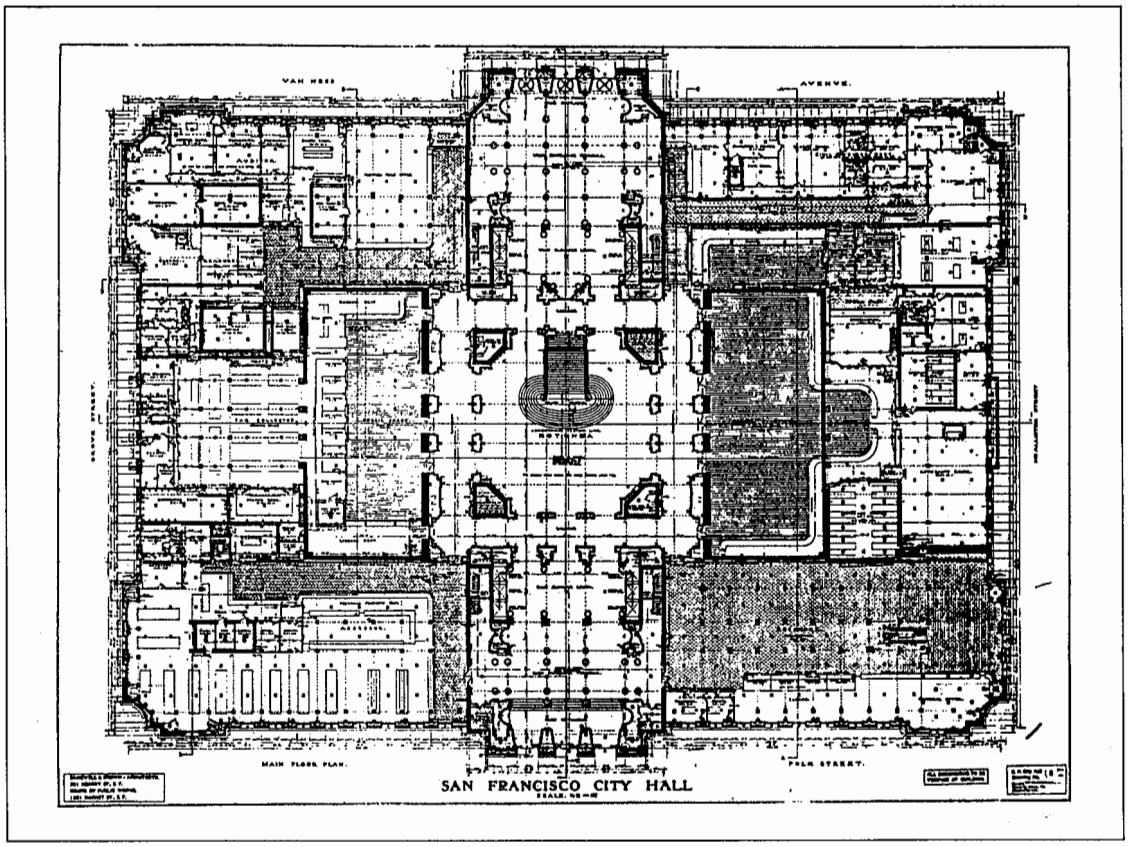

FIGURE 32. Plan of San Francisco City Hall.

centrated in the hollow clay tile and unreinforced masonry walls, and in concrete slabs.

The lateral load resisting system of the city hall building is composed of three major components:

- The unreinforced brick masonry walls;
- The hollow clay tile infill walls; and,
- The steel frame.

The extent of the damage to, and the required repair of, the building was evaluated in relation to requirements of the *San Francisco Building Code* (SFBC) Section 104 for seismic upgrade of the building structure.[4] The extent of the damage and the number of elements requiring repair exceed the threshold measure of a 30 percent tributary area to those affected vertical and lateral load resisting elements. Therefore, seismic upgrading of the structure to meet the current SFBC requirements was warranted.

Dynamic structural analyses were performed that acknowledged the complex interaction between lateral load resisting materials and systems as well as the interaction between the building and the dome. Load versus deformation curves were developed to not only investigate the interaction and response of the unreinforced masonry and hollow clay tile walls, but also to correlate the observed and calculated damage resulting from the Loma Prieta Earthquake.

The seismic strengthening will involve repairs to damaged unreinforced masonry and hollow clay tile walls and the construction of supplementary load resisting systems to stiffen and strengthen the building to withstand severe earthquakes without collapse. The supplemental systems should have a minimal impact on the building's historic aspects. Both fixed-base and base-isolation solutions were studied. Base isolation was chosen because it offered the following advantages:

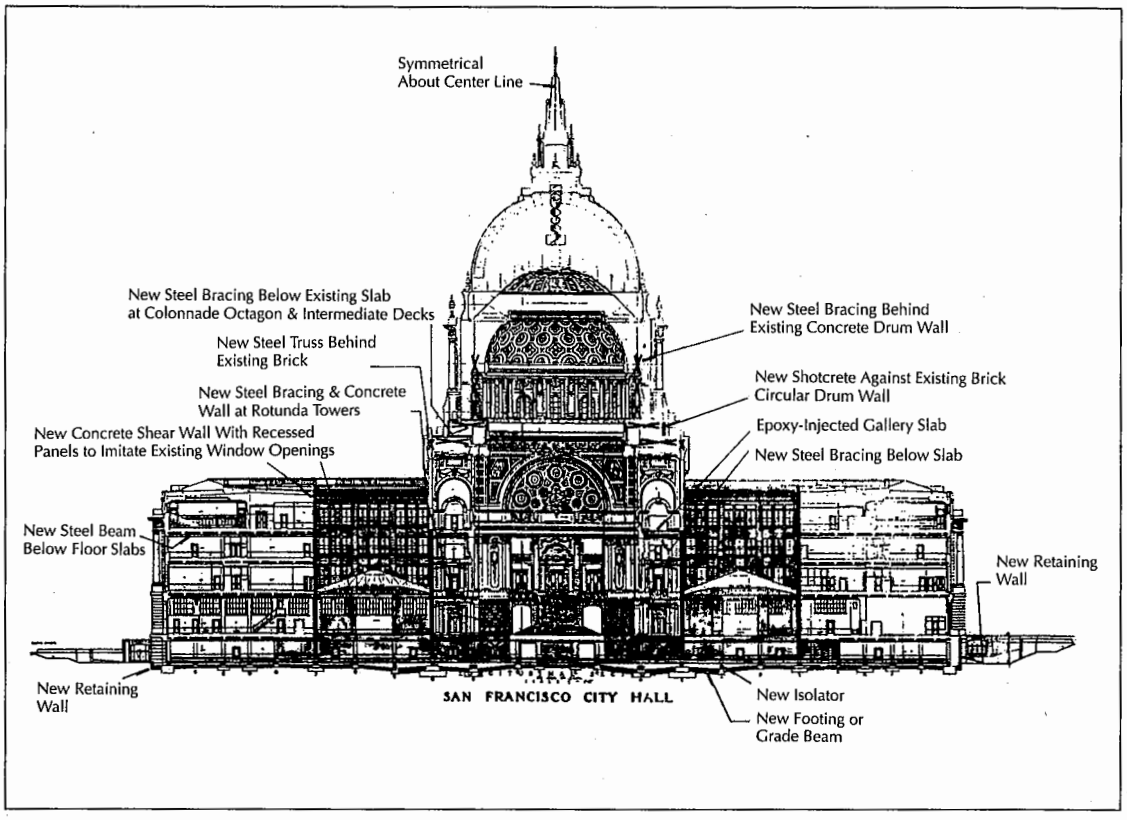

FIGURE 33. Longitudinal building section of San Francisco City Hall showing the base-isolation scheme.

- Significantly reduced seismic response;
- Minimum alteration of interior public spaces;
- Minimum disruption of important historic features;
- Minimum structural work on the dome;
- Minimum damage to architectural finishes during future earthquakes; and,
- Lower cost than a fixed-base solution.

A new ground floor constructed of steel framing supporting metal deck with concrete fill will be located above the isolators to distribute the lateral forces uniformly among the isolators (see Figure 33). New shear walls will be installed at the light courts to stiffen and strengthen the superstructure.

The seismic strengthening of this building is complex both technically and logistically because of the need to relocate personnel and functions now occupying the building. Completion of this project is scheduled for late 1997.

Conclusion

The case studies presented above serve to illustrate the diverse issues that must be taken into consideration in the seismic strengthening of existing buildings. Those examples are intended to demonstrate the great variety of solutions and methods that can be used as well as the complexity of analysis and detailing that may be necessary to accomplish the task.

It is hoped that as awareness of the need to seismically strengthen and preserve existing buildings increases, research and the creative ingenuity of the engineering profession will further enhance the palette of solutions and methods that can be used to accomplish this important task.

ACKNOWLEDGMENTS — *This article was first presented as part of the BSCES Structural Group's 1992 Fall Lecture Series. Dames & Moore performed the site-specific response spectra for the City and*

County Building in Salt Lake City. The Oakland City Hall Building was designed by the architectural firm of Palmer & Hornbostel. The San Francisco City Hall building was designed by the architectural firm of Bakewell and Brown. The structural analysis programs used for LLNL Building 111 were the ETABS three-dimensional finite element program for structures with rigid diaphragms and SAP90, a three-dimensional finite element program for modeling frames, shells and asolid elements.

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Predictions & Observations of Groundwater Conditions During a Deep Excavation in Boston

The uncertainty associated with predicting groundwater flow through bedrock in a deep excavation requires using a flexible design approach and a monitoring program.

CHRIS M. ERIKSON &
DAVID A. SCHOENWOLF

Several groundwater-related issues were investigated and documented in connection with the design and construction of a seven-level underground parking garage at Post Office Square in downtown Boston. Identifying and interpreting local hydrogeological conditions were critical to assessing the feasibility of a permanent underdrain system and its impacts on adjacent streets and buildings.

During the design phase, an extensive subsurface exploration program was undertaken to characterize subsurface hydrogeologic conditions. The program included installing observation wells and piezometers, permeability testing and a pump test program. Analytical and numerical dewatering simulation models were used to predict changes in groundwater conditions as the excavation progressed. Due to the lack of local experience associated with the unprecedented excavation depth, there were many uncertainties associated with the selection of hydrogeologic properties, model assumptions and boundary conditions.

During the construction phase, an extensive groundwater instrumentation monitoring program was implemented to measure groundwater response in the vicinity of the excavation.

Project Background

Post Office Square Garage consists of a seven-level below-grade parking structure with a public park located on the garage roof at

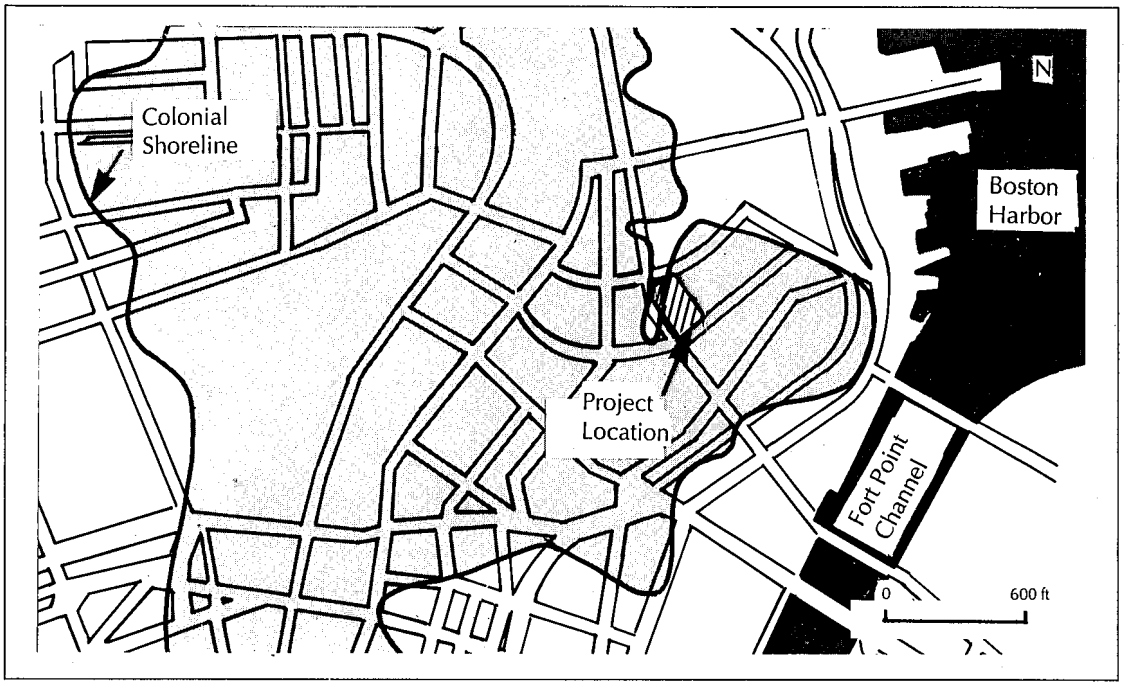

FIGURE 1. Project location.

ground surface. The project is located within Boston's downtown Financial District as shown in Figure 1, and is bounded by Milk, Congress, Franklin and Pearl Streets.

The project was built using the top-down construction method as described by Whitman *et al.*[1] and Schoenwolf *et al.*[2] The perimeter wall consisted of a cast-in-place concrete diaphragm wall (slurry wall) extending to a depth of 85 to 90 feet. A cross section through the garage is shown in Figure 2.

The subsurface soil and rock conditions (also indicated on Figure 2) from the ground surface downward are primarily composed of five strata.

There is a surficial layer of miscellaneous fill consisting of a fine sand to sandy gravel with varying amounts of brick, concrete and granite blocks. This fill varies in thickness from two to 13 feet.

Below the fill is a gray silty clay deposit, ranging in thickness from 35 to 50 feet with lenses and layers of sand and silt. The undrained shear strength of the clay varied from 1,000 to 1,500 pounds per square foot.

A sand layer, ranging in thickness from one to nine feet, was encountered at most locations around the site. This deposit can be described as medium dense to very dense, fine to coarse sand with varying amounts of fine to coarse gravel.

The glacial till underlying the site generally consisted of a hard, clayey to gravelly silt to a very dense silty coarse to fine sand, ranging in thickness from five to 38 feet. (Portions of this stratum have since been reclassified by Humphrey as glaciomarine deposits.[3])

Bedrock consisted of a very soft to moderately hard, completely to very slightly weathered argillite and sandstone.

The garage is surrounded by city streets. The nearest buildings range from 50 to 75 feet from the site perimeter. Foundations of these structures vary from spread footings supported in the upper clay stratum to deep caissons and piles installed into the glacial till. Numerous utilities are located below the adjacent streets in the fill stratum.

Two options were considered for the design of the lowest level garage floor slab:

- A structural mat capable of resisting full hydrostatic pressures; or,
- A slab-on-grade with hydrostatic pressures relieved by a permanent underdrain system below the slab.

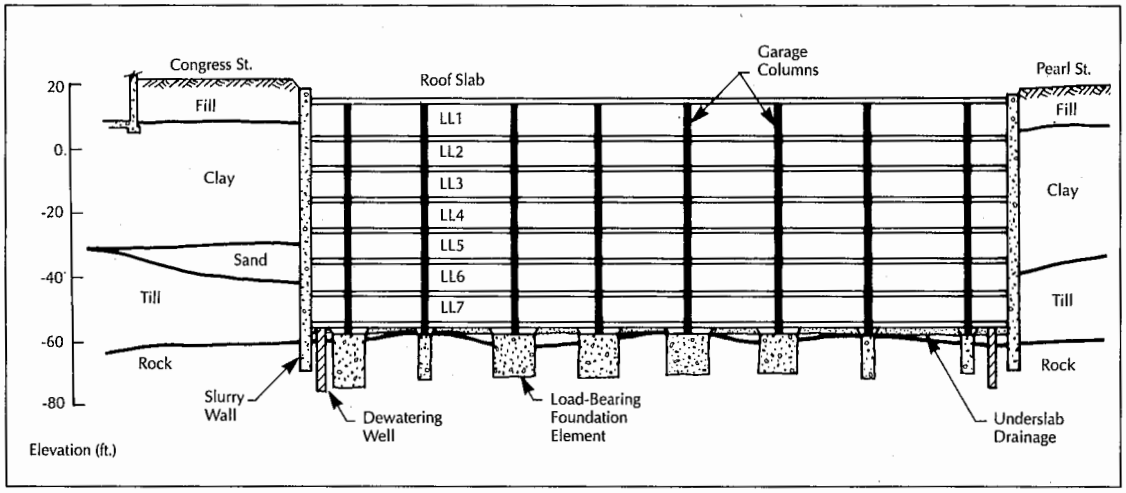

FIGURE 2. East-West cross section through the project.

In order to determine the feasibility of the latter option, major hydrogeologic engineering issues were investigated:

- What quantity of groundwater flow will enter the underdrain system, and how might this quantity (seepage) be minimized?
- What are the long-term effects of the underdrain system on the surrounding groundwater and piezometric levels? What would be the aereal extent of the potential drawdown?
- Due to reductions in piezometric levels, what would be the resulting settlement of the clay stratum and what would be the potential effects on adjacent buildings and utilities?

To evaluate these issues, in-situ permeability testing and a pump test program were conducted to estimate hydrogeologic parameters for analytical and numerical dewatering simulation models.

Subsurface Explorations

Two phases of subsurface explorations were undertaken for the project. The first phase consisted of 15 borings that were drilled within the site limits. These borings were primarily conducted for foundation design and included installing six shallow observation wells and eight piezometers. The groundwater instrumentation installed during this phase was for preliminary design purposes and was later destroyed when the old above-ground garage occupying the site was demolished. During this phase of the subsurface explorations, 14 falling head permeability tests were performed in the clay, sand, glacial till and bedrock strata. Four water pressure (packer) tests were also performed in the bedrock.

Supplementary explorations were undertaken primarily to install instrumentation for groundwater monitoring during construction. These instruments were installed approximately one year before the anticipated start of construction to establish preconstruction groundwater and piezometric levels. The instrumentation included seven observation wells and 34 piezometers that were installed within an approximate two-block area surrounding the site as shown in Figure 3. During this exploration phase, 35 additional variable head permeability tests (rising and falling head) were performed in the sand and glacial till strata and bedrock. However, these tests were primarily conducted within the bedrock in an attempt to better define permeability variations with depth below the top of rock.

Pneumatic and standpipe piezometers were installed in the completed boreholes. Pneumatic piezometers were primarily installed within the clay stratum and consisted of a diaphragm-acting porous stone. Standpipe pie-

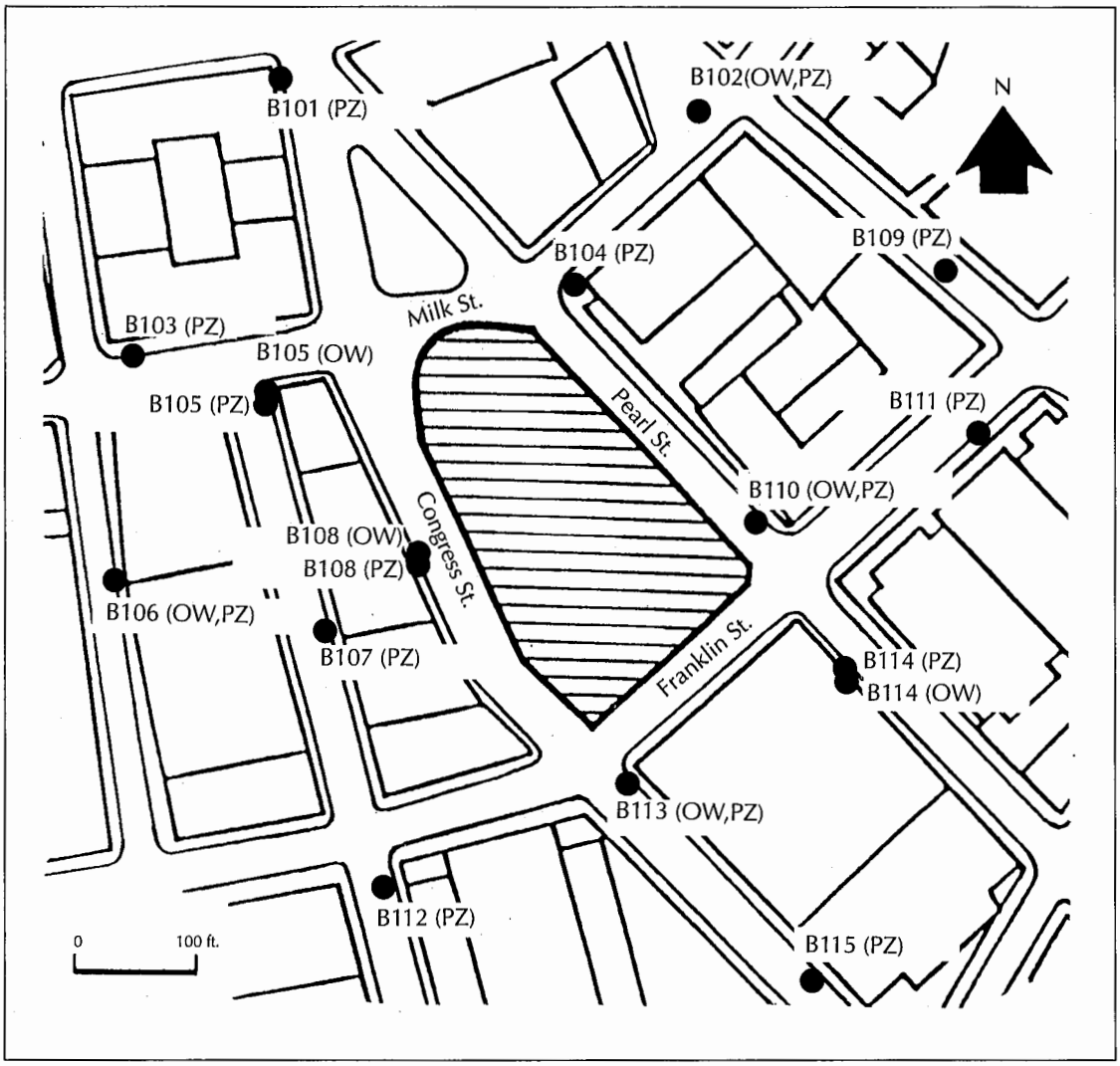

FIGURE 3. Location of the groundwater instrumentation.

zometers were typically installed in the more pervious glacial till and bedrock. The standpipe piezometer tips were 12 inches long, 1-½ inches in diameter, and had a screen consisting of a 40-micron vyon filter.

Figure 4 presents preconstruction groundwater and piezometric levels from both exploration phases. The figure indicates an apparent hydraulic gradient within the clay stratum. Observation wells installed in the fill and piezometers installed in the upper clay layer (at or above el. 0) indicated hydrostatic conditions with a corresponding groundwater level of approximately el. 10 (typical of this area). However, at or below el. -30, depressed piezometric levels were recorded in the lower clay layer, glacial till and bedrock, with an average piezometric level of approximately el. -8.

It is uncertain what conditions may have caused this variation. There was no apparent deep (bedrock) pumping near the project area.

Results from both phases of in-situ permeability tests are summarized in Figure 5. The results have been normalized to reflect the range of permeabilities as a function of depth below the top of bedrock. In general, the bedrock permeability ranged from 10^{-3} to 10^{-5} centimeters per second (cm/sec), with little to no trend with depth. The permeability of the gla-

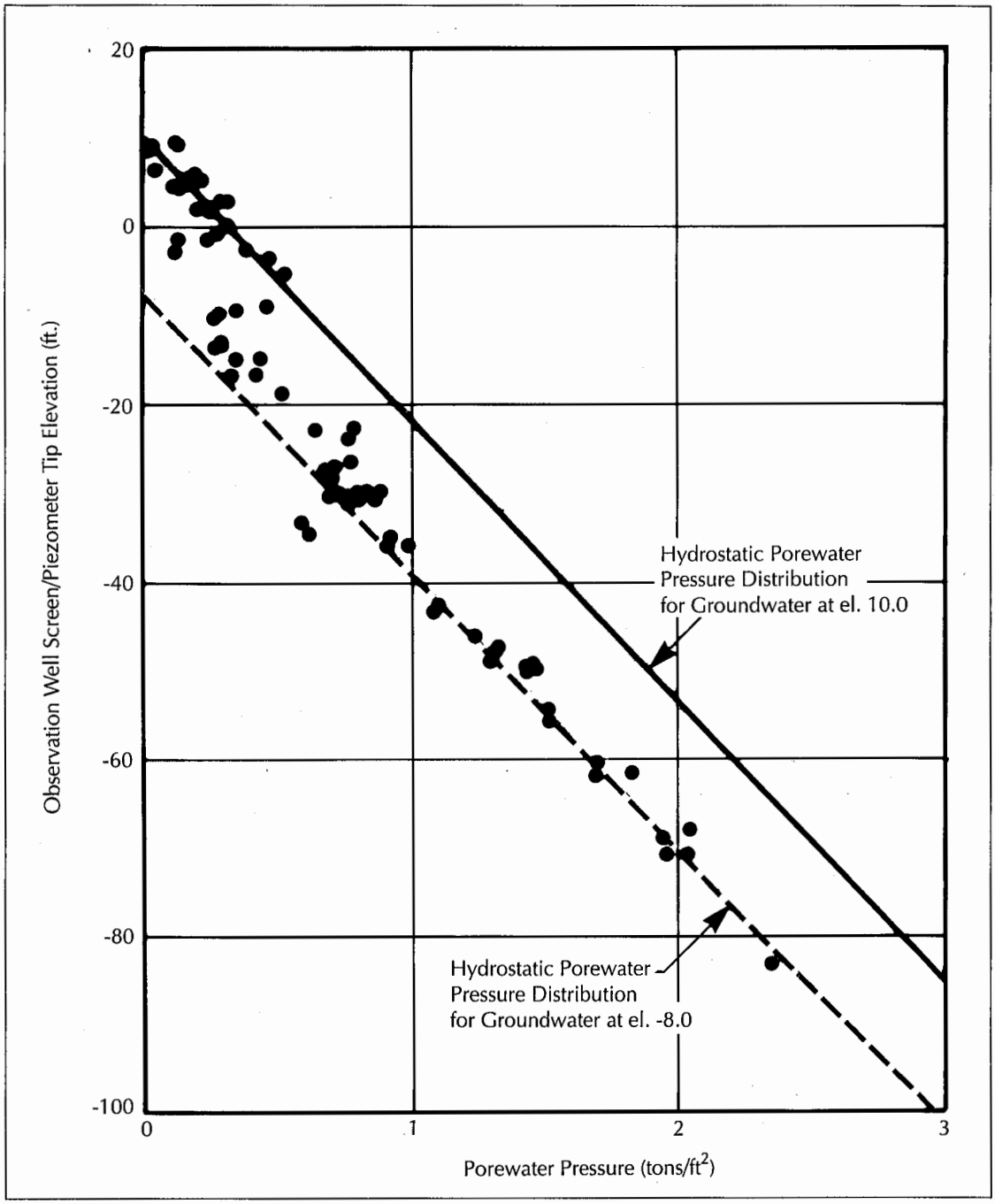

FIGURE 4. Preconstruction groundwater and piezometric levels.

cial till typically appeared to be less than the bedrock, varying from 10^{-4} to 10^{-6} cm/sec.

Also of interest were six tests within the bedrock in which both falling and rising head permeability tests were conducted. Four of the six tests indicated higher permeability values for the rising head tests. A comparison between the tests revealed that the results were within 0.05 cm/sec of each other.

Finite Element Analyses

Parametric analyses were performed on a mi-

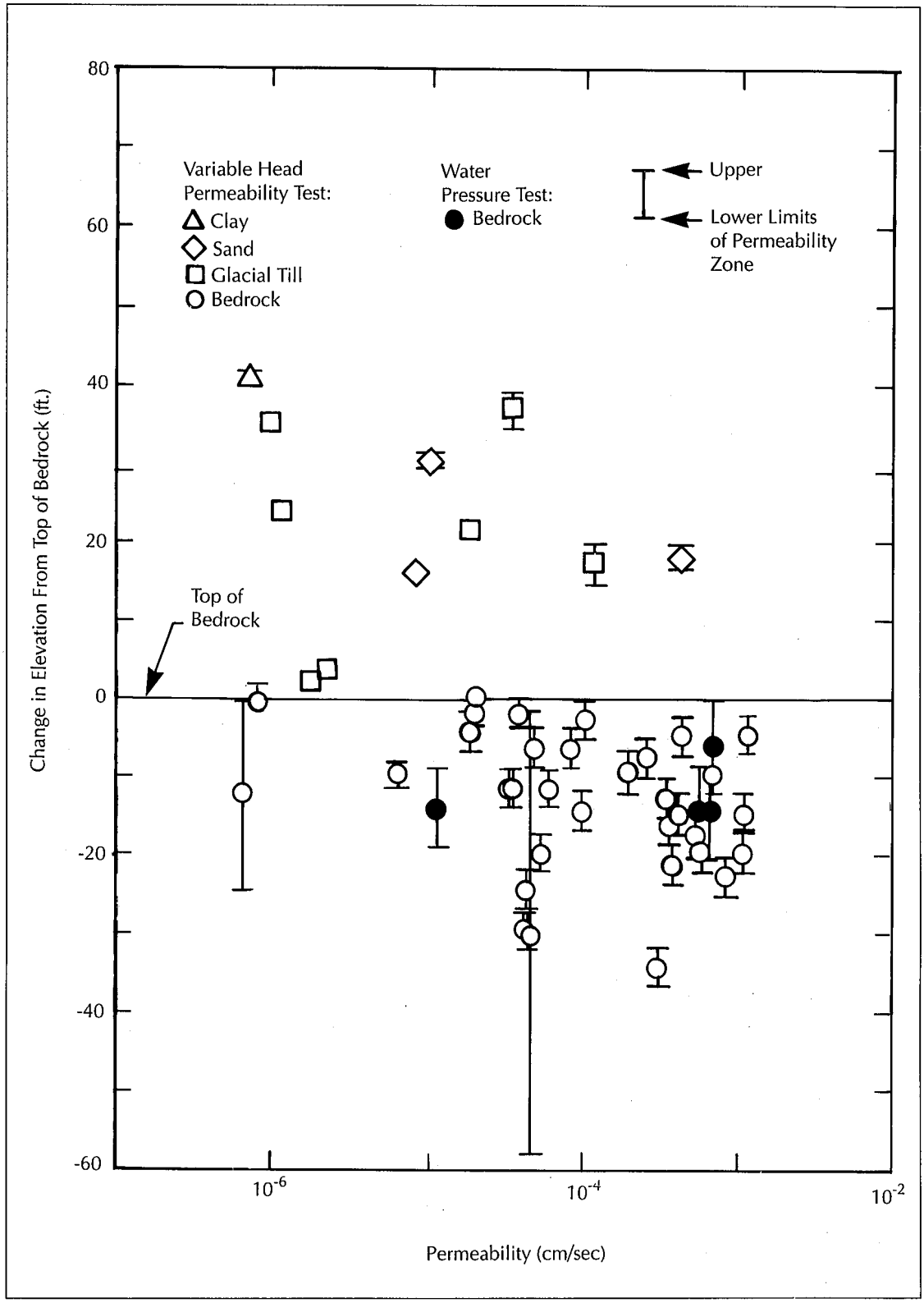

FIGURE 5. Summary of in-situ permeability tests.

cro-computer using finite element analysis software. The results of the finite element analyses were presented in flownet form using a graphical presentation software package.

The analyses were conducted to estimate the quantity of flow into the excavation and the potential drop in piezometric levels at the bottom of the compressible clay stratum. Table 1 summarizes the assumptions associated with the analyses.

From the results of the analyses, several conclusions were drawn. Theoretical estimates of groundwater flow into a permanent underdrain system depend to a high degree on the in-situ permeability of the bedrock and the assumed boundary conditions for the particular analysis. For the average value of bedrock permeability (see Table 1), the estimated total flow into the underdrain system was approximately 20 gallons per minute (gpm). For the highest assumed bedrock permeability, the total estimated flow was approximately 250 gpm.

Increasing the slurry wall penetration below the bottom of the excavation minimally reduces flow into the excavation. The greatest reduction of flow with increasing slurry wall depth occurred for the case of lowest bedrock permeability. For a constant total head boundary at the bottom of the bedrock stratum, the depth of slurry wall cutoff had the least influence on the flow quantity.

Less piezometric head loss and higher seepage quantities were calculated when assuming a constant total head at the bottom bedrock boundary as compared to an impervious boundary.

The decrease in piezometric head at the base of the clay stratum varied considerably, based on the assumed bedrock permeability and analysis boundary conditions. For the analysis that considered the highest rock permeability and an impervious bedrock bottom boundary, a total piezometric head loss was estimated within 50 feet of the excavation (zero piezometric head at the bottom of the clay). Alternatively, minimal piezometric head loss was estimated at the bottom of the clay when the lowest bedrock permeability and a constant head boundary was assumed.

TABLE 1
Summary of the Finite Element Analyses

Excavation Support/Cutoff System Details:

- Ground Surface: el. 20
- Bottom of Excavation: el. -55
- Excavation Support System:
  Reinforced Concrete Diaphragm Wall (Slurry Wall):
  Thickness: 3.0 ft.
  Depth of Embedment Below Bottom of Excavation Varied From 5 to 30 ft.

Soil Profile & Permeabilities

Subsurface Stratum	Thickness of Stratum (ft.)	Permeability (cm/sec)
Fill	15	1×10^{-5}
Clay	40	7×10^{-7}
Sand	10	1×10^{-5}
Glacial Till	15	2×10^{-6}
Bedrock	>135	Low Est.: 7×10^{-7}
		Ave. Est. 5×10^{-5}
		High Est.: 7×10^{-4}

Piezometric Boundary Conditions:

- Hydrostatic Groundwater Level: el. 10
- Bottom Bedrock Boundary: el. -195 With Two Conditions Analyzed:
  1) Impervious;
  2) Constant Head Boundary With Hydrostatic Level at el. 9

Pump Test Program

Three deep bedrock dewatering wells were installed and tested as part of an on-site pump test program. The wells were installed through approximately 100 feet of overburden soils and screened approximately 40 feet into bedrock. The well screens consisted of six-inch diameter wire wound screen. Observation wells and piezometers (as shown in Figure 3) were monitored during the pump test. Specific details of the pump test program are presented in Table 2. Results of the pump test and subsequent analytical and numerical simulation models indicated the following:

- Assuming a saturated aquifer thickness

TABLE 2
Summary of the Pump Test Program

Test Well No.	Bedrock Screen Range (el.)	Duration of Pumping (Days)	Pumping Rate (gpm)	Static Piezometric Level Prior to Tests (el.)	Drawdown at End of Pumping (el.)
TW-1	-75 to -115	5	5.9	-8.3	-51.0
TW-2	-76 to -118	4	3.4	-8.5	-84.0
TW-3	-78 to -118	5	0.9	-7.1	-94.8

approximately equal to the 40-foot well screen length, the bedrock permeability values ranged from 1×10^{-4} to 9×10^{-4} cm/sec.

- Although several factors may reduce the actual long-term flow into the excavation, it was determined that the underdrain system should be designed to accommodate permanent flows of up to 100 gpm.
- The major source of flow into the excavation would be through the bedrock. The slurry wall cutoff will serve to decrease the flow from the overburden soils into the excavation. However, the wall provides little effect in decreasing the flow from the bedrock.

Ground Settlement Due to Dewatering

As piezometric head losses develop within the clay stratum, consolidation of the clay occurs, resulting in ground surface settlement. These settlements are typically proportional to the clay stratum thickness and the magnitude of head loss within the clay stratum, which theoretically decreases with distance from the excavation.

Long-term ground surface settlements due to dewatering were calculated based on the piezometric head losses estimated from the finite element analyses. The head losses were calculated based on assuming an impervious bottom boundary in the bedrock and the low and average bedrock permeabilities as indicated in Table 1. The stress history of the clay deposit, as estimated from consolidation tests, indicated that the increased effective stress in the clay caused by dewatering would not exceed preconsolidation values. The estimated settlements ranged from 1-⅜ inches for locations nearest the site, where the clay stratum was observed to be the thickest; to ¼ inch at locations greater than 75 feet from the site, in areas where the clay layer is thinner.

The estimated long-term settlements were calculated assuming hydrostatic conditions through the clay layer. However, as indicated in Figure 4, depressed piezometric levels were measured at the bottom of the clay layer. It was not certain how long these levels were depressed. If depressed piezometric levels were assumed in the settlement analysis, settlements were estimated to be ¼ to ½ inch less than those calculated assuming hydrostatic conditions.

Groundwater Control Considerations

Using the results of the hydrogeologic studies and settlement analyses, an approach to groundwater control was adopted. It was generally concluded that significant uncertainty exists in quantifying the bedrock permeability. Although the bedrock was modeled as a homogeneous porous media, seepage quantities into the excavation could locally vary significantly depending on bedrock joint spacing, width and orientation. In order to assess this uncertainty during construction, an extensive groundwater instrumentation monitoring program was undertaken. Piezometric levels were to be monitored both inside and outside the excavation limits.

Settlement predictions of adjacent structures, due solely to consolidation of the clay,

were estimated to be tolerable. For several adjacent buildings located within 50 feet of the excavation, "worst case" foundation settlements of ¾ inch or less were estimated.

For the permanent underdrain system alternative, consideration had to be given to:

- The quantity of flow the present and future owners would be willing to provide for and maintain; and,
- The amount of discharge from the system that the associated public agency would determine to be acceptable.

Since the estimated flow quantity was relatively uncertain, the substructure design included provisions for installing either a permanent underdrained slab or a structural slab that could resist full hydrostatic uplift pressures. Economically, the permanent underdrain system was the preferred alternative. However, if during construction the quantity of flow into the excavation was deemed to be excessive, a pressure slab could be constructed with minimal delay.

Studies indicated that extending the bottom of the slurry wall deeper into the bedrock stratum did not significantly reduce groundwater flow into the excavation. Therefore, the slurry wall was typically installed at least two feet into rock. For contingency purposes, four-inch diameter PVC pipes were installed in slurry wall panels at ten-foot spacing to allow access for post-grouting below the slurry wall, if necessary. A grouting program was to be considered if isolated areas of seepage were identified below the bottom of the wall. It was envisioned that grouting would potentially decrease seepage quantities into the excavation.

Construction Dewatering Program

To maintain stable subgrades during the excavation sequence, a temporary dewatering program was implemented. The program consisted of ten dewatering wells in addition to the three existing test wells. Locations of the wells are shown in Figure 6.

Seven of the ten wells were advanced using a 17-inch diameter roller bit. The completed wells consisted of an eight-inch diameter riser pipe with screen (0.040-inch louvered) installed through the sand, glacial till and bedrock. After installing the well screen, the annular space around each well screen was backfilled with a filter-compatible clean processed gravel.

The remaining three dewatering wells (W2, W6 and W9) were installed using a three-foot wide by eight-foot long clamshell bucket excavated under a biodegradable polymer slurry ("trench" wells). The riser pipe and screen were 24 inches in diameter.

In an attempt to limit the amount of piezometric drawdown outside the project limits, the rotary-drilled wells were typically installed ten feet into rock and generally no greater than two feet below the bottom of the slurry wall (el. -70). Wells W2, W6 and W9 were installed to the lowest excavation level, approximately el.-55.

Pumping from the wells was conducted on a continuous basis that began during the excavation of the first subgrade level. The number of active wells and the pumping level within each well varied as the depth of the excavation advanced. To monitor the effectiveness of the pumping, nine vibrating wire piezometers were installed at selected locations — two in sand, one in glacial till and six in bedrock. The locations of these piezometers are also shown in Figure 6.

Results of the Construction Monitoring Program

As indicated in Figure 7, construction dewatering was initiated in August 1989 and continued until the end of June 1990. Initially, one well was used to dewater the upper excavation levels. As the excavation deepened, the number of wells in operation gradually increased to nine. Total pumping rates for all wells in operation during construction varied from three to 16 gpm. Groundwater collected during construction dewatering was pumped into a settling tank prior to discharge. Minimal fines or sediments were observed in the settling tank during dewatering operations.

In July 1990 the permanent underdrain system was installed. This system consisted of a series of six-inch diameter perforated PVC pipes embedded in a 12-inch thick layer of ¾-inch stone. Filter fabric was laid on the exposed subgrade prior to stone placement. Total flows into the underdrain system and meas-

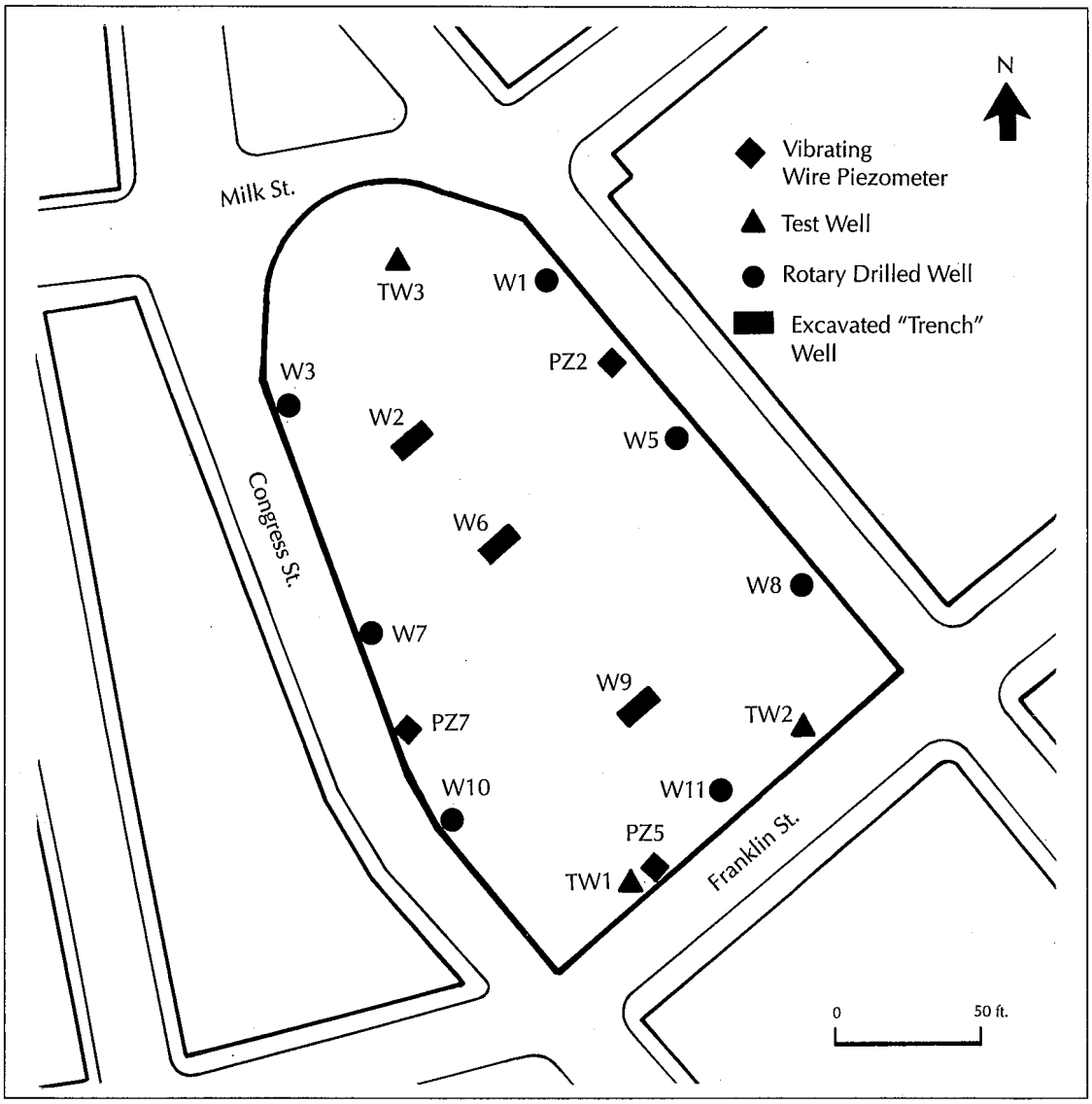

FIGURE 6. Location of the construction dewatering wells and piezometers.

ured in the sump pit ranged from eight to 12 gpm.

The results of the groundwater instrumentation monitoring outside the site are presented in Table 3. The finite element analyses performed during design had indicated that the greatest drawdown would occur in the bedrock. This drawdown was confirmed at B110-C, where a maximum drawdown of 37 feet was observed (see Figure 8). It appears that the magnitude of drawdown indicated by the bedrock piezometers may have been affected by the location of the tips since the deeper piezometers showed the greatest amount of drawdown.

Maximum drawdown in the glacial till and clay during construction was seven and five feet, respectively. Drawdown in the clay was typically minimal and appeared to depend more on the distance of the piezometer tip from the bottom of the clay layer than the distance of the piezometer from the excavation (see Table 3). Virtually no changes in groundwater levels were observed in the fill during construction.

Figures 8 and 9 present typical piezometric response to construction dewatering at various

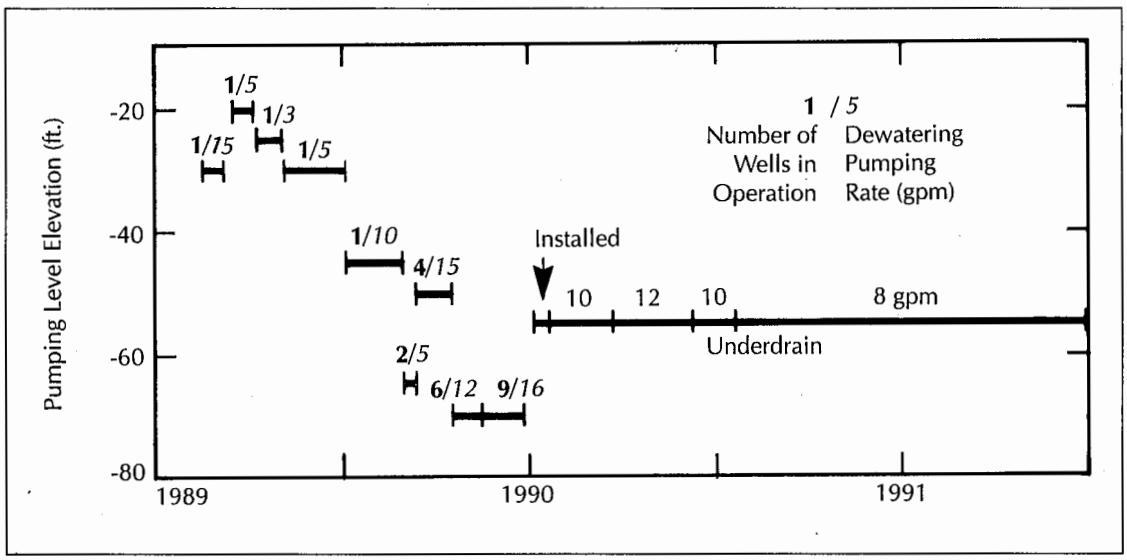

FIGURE 7. Time line of construction dewatering.

TABLE 3
Summary of Groundwater Instrumentation for the Post Office Square Garage Project

Piezometer/ Observation Well No.	Tip Location (el.)	Subsurface Stratum	Approximate Distance From Excavation (ft.)	Pre-Construction Level 10/88 (el.)	Maximum Level 6/90 (el.)	Level 12/91 (el.)	Maximum Drawdown 6/90 (ft.)	Drawdown 12/91 (ft.)
B110-OW	5.5	Fill	30	11	11	11	—	—
B113-OW	4.3	Fill	55	9	9	9	—	—
B102-OW	4.4	Fill	225	8	8	8	—	—
B106-OW	6.1	Fill	280	8	8	8	—	—
B110-PZ-A	0.3	Clay	30	11	10	11	1	—
B110-PZ-B	-18.7	Clay	30	-3	-10	-8	7	5
B108-PZ-A	-16.5	Clay	45	-3	-6	-3	3	—
B110-PZ-B	-31.0	Clay	45	-6	-17	-11	11	5
B104-PZ-A	-27.4	Clay	50	-5	-9	-6	4	1
B113-PZ-A	-10.2	Clay	55	-2	-2	-2	—	—
B114-PZ-A	5.9	Clay	115	12	10	(Destroyed)	2	—
B105-PZ-A	-1.4	Clay	130	6	6	6	—	—
B105-PZ-B	-16.4	Clay	130	-5	-8	-5	3	—
B107-PZ-A	-30.9	Clay	140	-5	-9	-5	4	—
B112-PZ-A	-9.5	Clay	215	-1	-3	-1	2	—
B112-PZ-B	-30.5	Clay	215	-8	-13	-10	5	2
B111-PZ-A	9.7	Clay	220	14	12	14	2	—
B102-PZ-A	-9.1	Clay	225	5	2	2	3	3
B103-PZ-A	-13.6	Clay	240	-4	-7	-7	3	3
B101-PZ-A	-30.9	Clay	250	-3	-5	-3	2	—
B115-PZ-A	-2.7	Clay	275	2	2	2	—	—
B113-PZ-B	-30.2	Sand	55	-6	-9	-6	3	—
B114-PZ-B	-23.6	Glacial Till	115	1	-4	(Destroyed)	5	—
B105-PZ-C	-48.9	Glacial Till	130	-8	-10	-9	2	1
B107-PZ-B	-43.4	Glacial Till	140	-8	-15	-12	7	4
B112-PZ-C	-47.5	Glacial Till	215	-8	-12	-9	4	1
B111-PZ-B	2.2	Glacial Till	220	9	(Destroyed)		—	—
B102-PZ-B	-28.1	Glacial Till	225	-5	-12	-12	7	7
B103-PZ-B	-36.1	Glacial Till	240	-7	-7	-7	—	—
B101-PZ-B	-55.4	Glacial Till	250	-7	-7	-7	—	—
B115-PZ-B	-13.2	Glacial Till	275	-5	-5	-5	—	—
B109-PZ-A	3.3	Glacial Till	280	12	12	12	—	—
B109-PZ-B	-3.2	Glacial Till	280	11	11	11	—	—
B106-PZ-A	-34.4	Glacial Till	280	-15	-15	-15	—	—
B110-PZ-C	-83.2	Bedrock	30	-8	-45	-40	37	32
B108-PZ-C	-71.0	Bedrock	45	-8	-36	-26	28	18
B104-PZ-B	-70.9	Bedrock	50	-5	-30	-25	25	20
B113-PZ-C	-62.2	Bedrock	55	-8	-16	-12	8	4

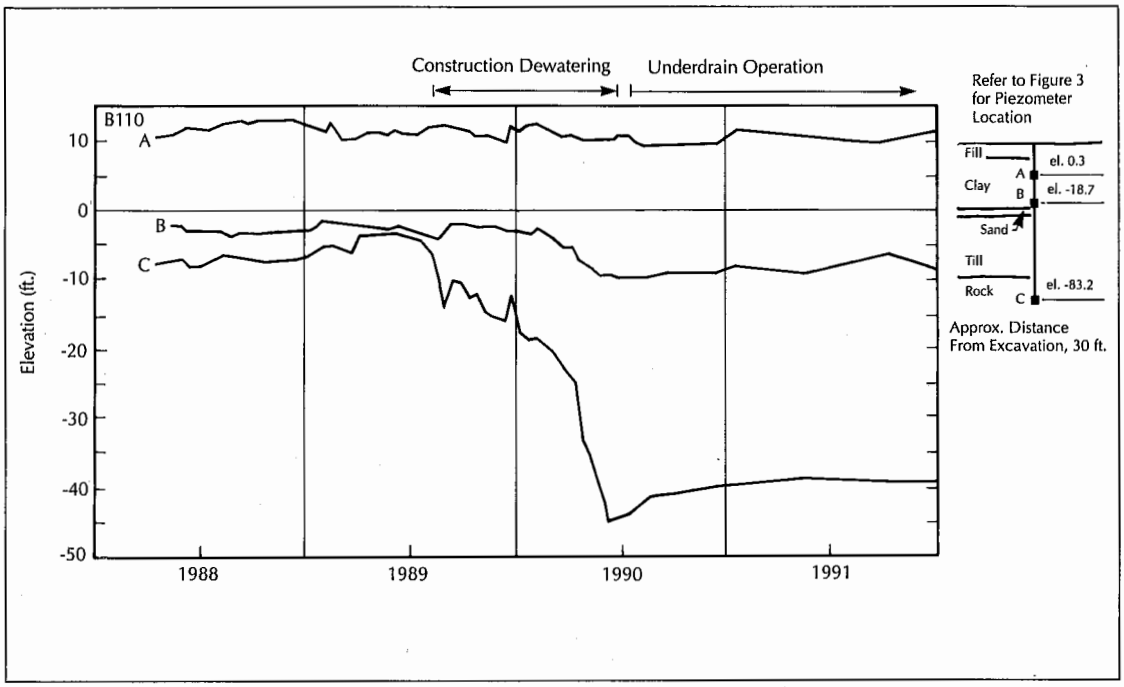

FIGURE 8. Piezometric head versus time for B110.

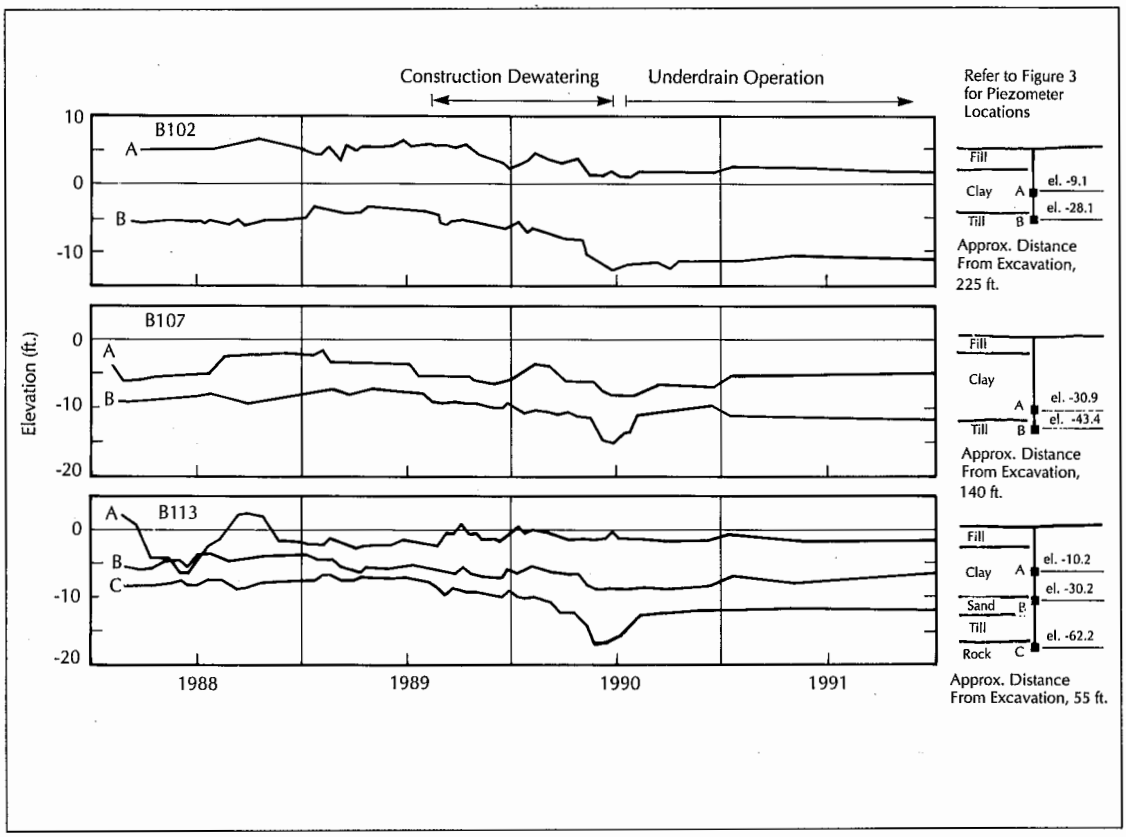

FIGURE 9. Piezometric head versus time for B102, B107 and B113.

instrumentation locations. The piezometers shown were selected to illustrate the effects of the varying subsurface conditions surrounding the site.

Piezometers located in boreholes B102 and B107 represented the subsurface conditions northeast and west of the site where the clay stratum directly overlies glacial till. As indicated in Figure 9, piezometers located in the clay and glacial till responded to construction dewatering and data approximately paralleled each other.

Piezometers located in boreholes B110 and B113 represented subsurface conditions at the southern end of the site where a sand layer existed between the clay and glacial till strata. The data collected at these piezometers (see Figures 8 and 9) show a significant response to construction dewatering in the bedrock. However, due to the relatively pervious sand layer and the less pervious glacial till below, proportionally less head loss was observed within the clay. It is likely that the sand layer served to provide recharge to the bottom of the clay layer and, thus, reduced drawdown in this stratum.

Only three of nine piezometers installed within the site provided continuous data throughout construction as shown in Figure 10. The other piezometers were destroyed or temporarily obstructed or disabled during excavation operations.

The data presented in Figure 10 indicate the relatively rapid response of piezometric levels within the site with increased dewatering effort. However, during some periods of constant construction dewatering, drops in piezometric levels within the site appeared to have been related to increased excavation depth. This behavior was generally observed during the last three levels of excavation in mid-1990 when the level of dewatering within the site remained constant at approximately el. -70. These data would suggest that a relatively effective cutoff was provided by the slurry wall.

Figure 10 also indicates piezometric levels of up to eight feet greater than the existing subgrade level during May/June 1990 at piezometer PZ2. Although the elevated piezometric level at this location was of concern, no subgrade instability was observed and the piezometric head dropped during the next stage of excavation. Upon completion of the excavation, all three piezometer locations indicated piezometric levels above the subgrade level, reflecting seepage gradients below the underdrain system.

To relate the effects of groundwater drawdown with adjacent ground surface settlements, nine surface reference points were selected from points that were installed to monitor settlements near the site. The selected points are shown in Figure 11 and were installed prior to construction in September 1988 and were monitored until mid-1990. The nine points were selected since they were located near the corners of the slurry wall and were assumed to be essentially independent of slurry wall-related movements. The points were also located at least 100 feet from the excavation.

Measurements taken as of July 1990 indicated negligible movements at points 24, 42 and 62. Settlements at points 1, 2, 3, 22, 41 and 54 were approximately ¼ inch. These recorded settlements were generally within preconstruction estimates.

The maximum settlement for an adjacent building was ½ inch for a structure located approximately 50 feet from the excavation. Settlement of this structure was believed to be primarily related to lateral movements of the slurry wall.

Conclusions

Significant uncertainty exists in estimating the effects of groundwater control during the construction of a deep excavation. This uncertainty is typically related to the inability to accurately determine subsurface soil and rock permeabilities. For the Post Office Square Garage project, this uncertainty was compounded since the bottom of the excavation cutoff was embedded in bedrock.

Unlike the determination of flow through porous media, bedrock permeabilities may be significantly influenced by joint spacing, width and orientation. In addition, boundary conditions for analytical and numerical modeling are difficult to determine due to the potentially extensive depth of bedrock aquifer.

In general, the effects of groundwater control on excavations is directly related to specific

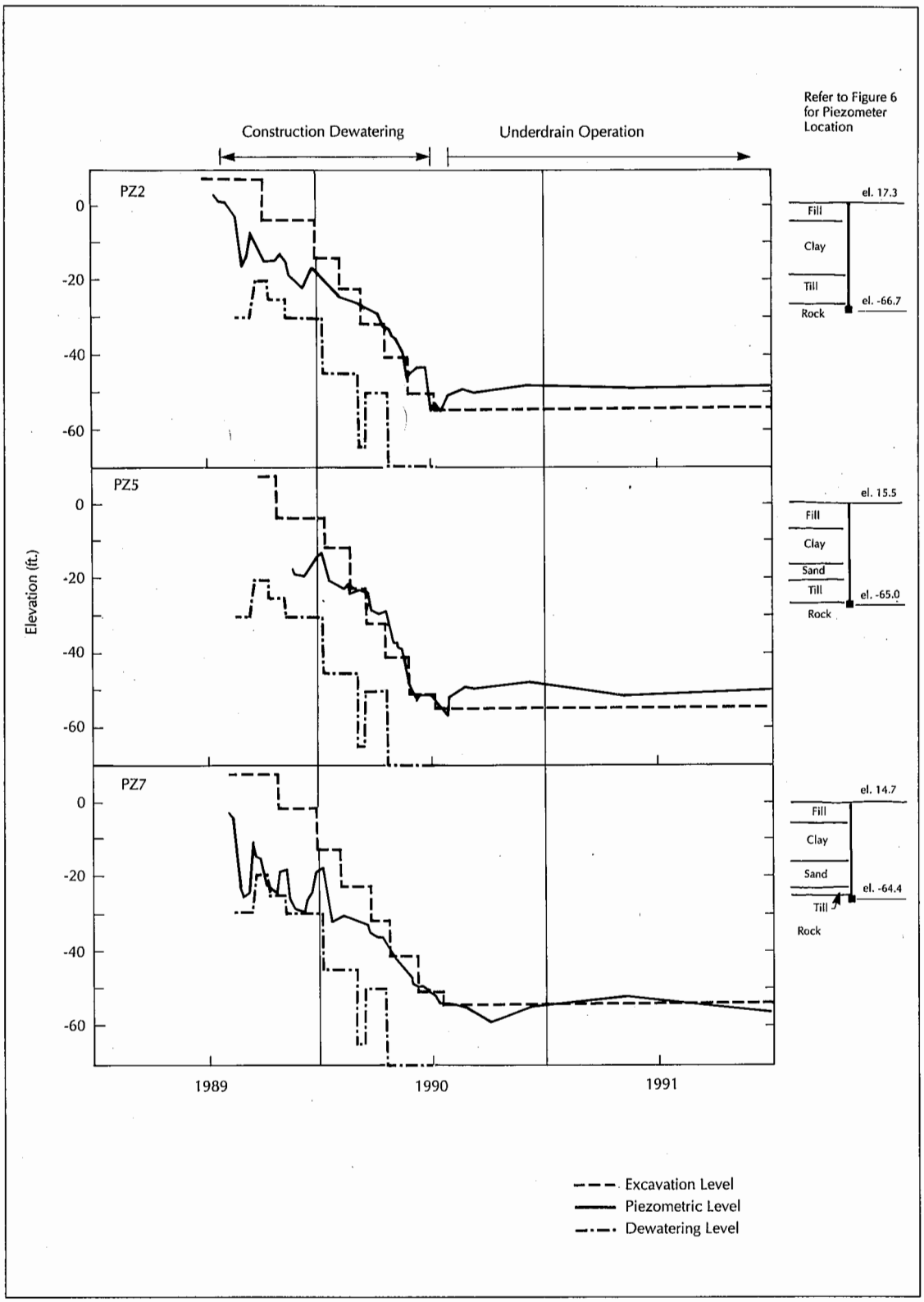

FIGURE 10. Excavation, piezometric and dewatering levels versus time within the project site.

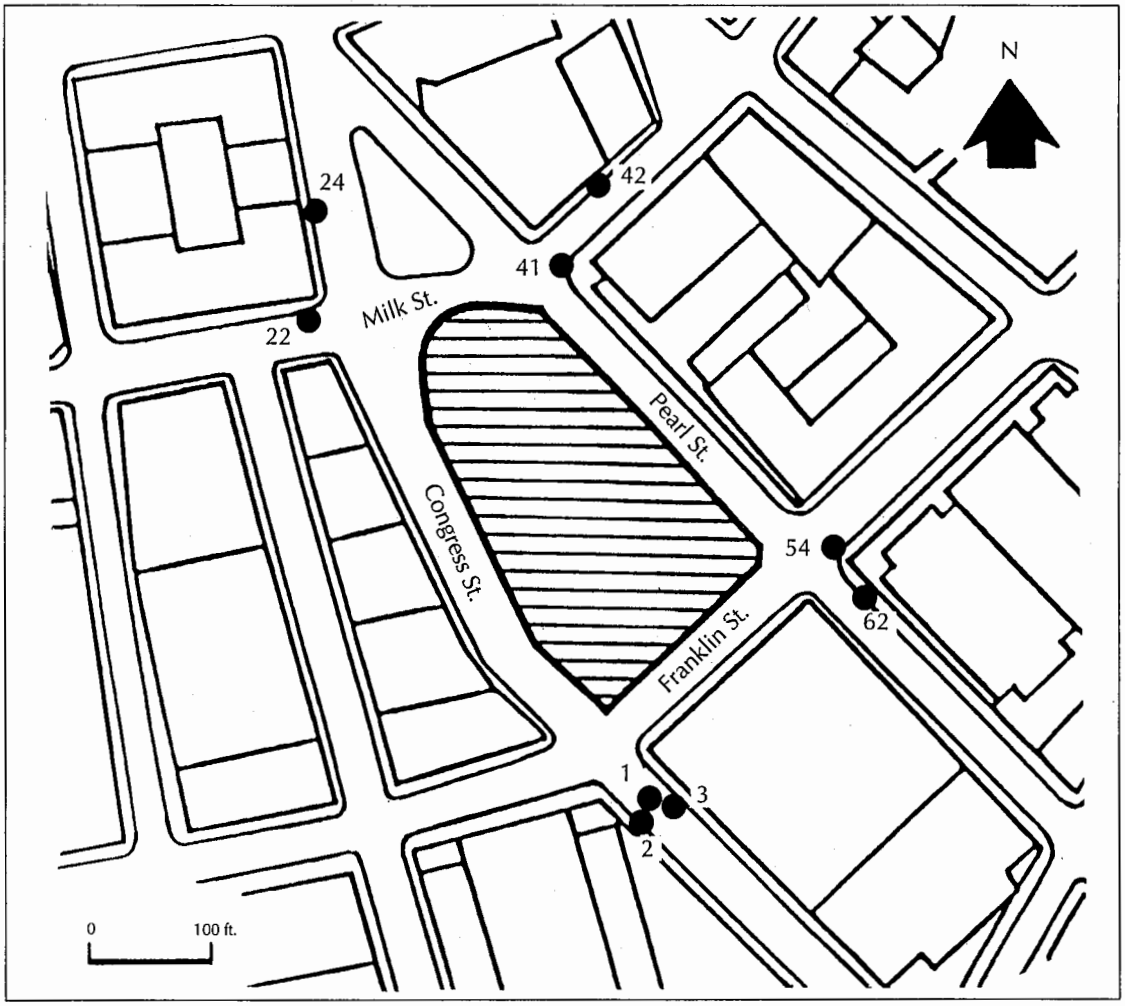

FIGURE 11. Selected surface reference points.

site details and should be investigated on a case-by-case basis for each excavation. Such details may include:

- The relationship between the depth of cut off and subsurface conditions;
- Areas of potential groundwater recharge;
- Adjacent foundation types; and,
- Existing structures that may act as conduits of groundwater.

As indicated by this project, a monitoring program should be developed to evaluate design assumptions for situations that contain significant uncertainty. Contingencies can then be developed that can be effectively implemented if the conditions that are encountered during construction vary significantly from those that have been assumed.

NOTES — *All elevations referred to here are referenced to Boston City Base Datum (BCB). Finite element analysis software used for the project was FLOWNET; the graphical presentation software was FLOWPLOT.*

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Launching Gantries for Bridge Erection in Difficult Terrain

In areas with limited ground access, the use of an overhead launching gantry may offer an economical solution for the construction of segmental concrete bridges.

W. SCOTT MCNARY & JOHN HARDING

Precast segmental bridges are being built in greater numbers and under increasingly difficult site conditions across the United States. These structures are often the last and sometimes most difficult sections of the Interstate highway system to complete. Overhead launching gantries provide a practical and environmentally sensitive solution to constructing bridges in areas where ground access is limited. These specialized pieces of equipment allow precast balanced cantilever construction entirely from above the structure, even under difficult geometric restraints such as urban alignments and tight radius curves.

Methods of Segmental Erection

The first precast concrete segmental bridge was erected in France under the direction of Eugene Freyssinet between 1945 and 1948. The bridge crosses the Marne River, located near Paris. In 1956 the first segmental bridge in the United States was erected near Shelby, New York.

There are three main configurations of segmental concrete bridges that are generally identified by the method of erection:

- Span-by-span;
- Incremental launching; and,
- Balanced cantilever.

Span-by-Span Erection. This type of bridge is erected one span at a time on a supporting structure. Post-tensioning is placed at midspan as well as over the piers in order to make the structure continuous.

The span-by-span erection method utilizes an underslung or assembly truss that supports concrete segments between two piers. Segments are placed on the truss using a crane. After all of the segments have been placed, post-tensioning tendons are installed and stressed so that the segments are self-supporting. The truss is then moved ahead to the next span. The truss may be launched or rolled ahead to the next span. In some cases, the truss may have to be partially disassembled, lowered to the ground and then moved ahead. After adjacent spans have been erected, post-

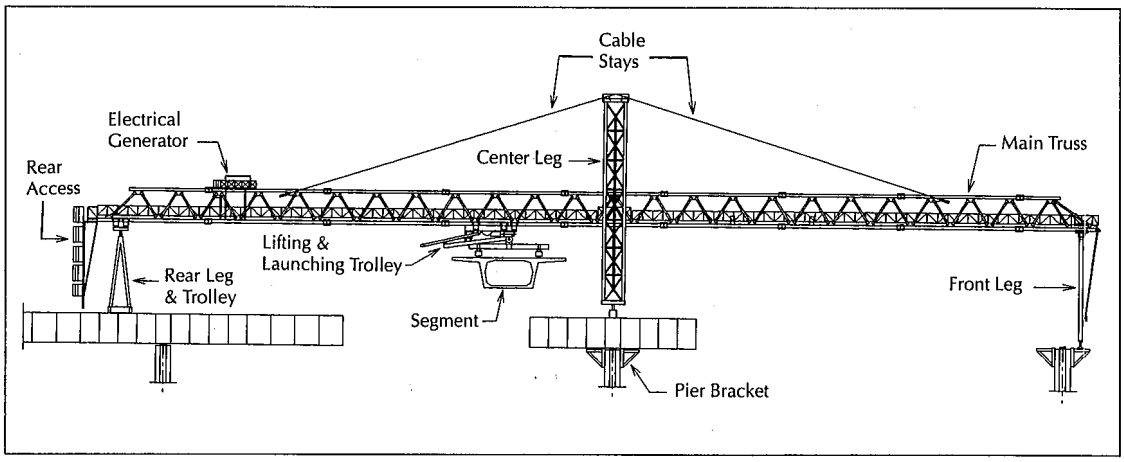


FIGURE 1. Schematic of a fixed-leg cable-stayed gantry.

tensioning is placed over the pier, making the two spans continuous.

The span-by-span method enables erecting the structure completely from above, which is especially important in difficult terrain or where ground access is limited. Due to economic considerations, span-by-span erection is typically used on structures with span lengths less than 150 feet. The method is also limited to structures with straight or large radius curved alignments.

Incremental Launching. The incremental launching method was developed in the early 1960s by two German engineers. This method involves setting up a casting bed at one of the abutments, where segments are match cast against a previously poured segment. Once the segment in the forms has reached sufficient strength, it is post-tensioned to the ahead segments and pushed or launched towards the ahead pier. This sequence continues until the structure reaches the opposite abutment.

The incremental launching method is limited to constant depth sections as well as constant radius curves. It has been successfully used on spans up to 200 feet without the need for temporary bents. The longest span built utilizing temporary bents was 550 feet and the longest structure has 23 spans with a total length of 3,400 feet. This method is a viable alternative for structures built in difficult terrain.

Balanced Cantilever. This type of bridge is built by alternately placing segments on either side of a supporting pier, thus forming a balanced cantilever. Negative moment post-tensioning is stressed during erection and positive moment post-tensioning is placed at mid-span to make the structure continuous with the previously completed structure.

Cantilever post-tensioning is placed and stressed during erection. When the cantilever is complete, a closure pour makes it continuous with the previously completed structure. Continuity post-tensioning is then placed at mid-span between the two structures.

There are three primary ways to erect a balanced cantilever bridge using:

- Crane;
- Beam and winch; or
- Launching gantry.

A land-based or barge-mounted crane can be used to erect segments. This method is suitable and versatile at sites where ground or water access is not a problem.

The beam and winch method involves supporting a winch on fixed beams, which are mounted to the leading edge of the completed portion of the cantilever. Segments are transported and positioned directly below the end of the cantilever. They are then lifted using the winch assembly and secured to the previously erected segment. Typically, one beam and winch setup is located at either end of the cantilever. As segments are placed, cantilever post-tensioning is installed and stressed. The beam

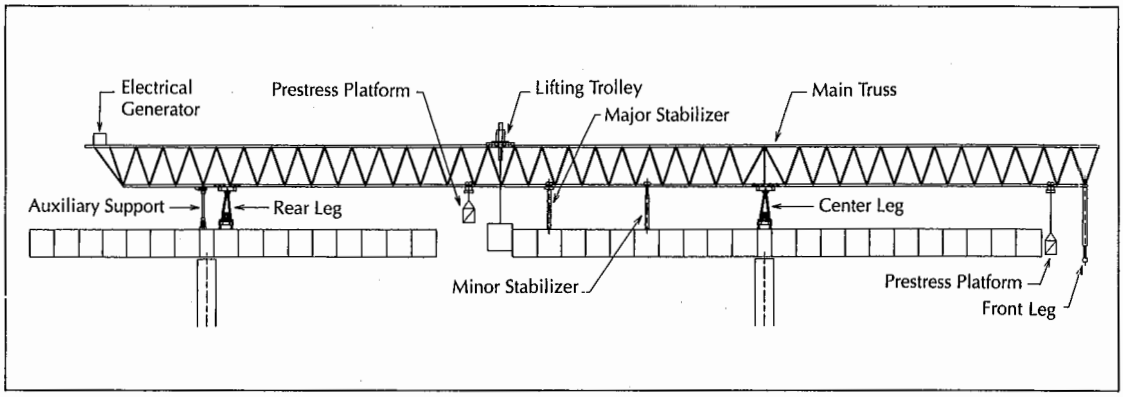


FIGURE 2. Schematic of a moveable-leg gantry.

and winch setup is then moved ahead to the end of the cantilever and positioned to lift the next segment. This versatile method can be used with virtually any alignment configuration. However, ground or water access is required to position segments directly below the lifting equipment.

The final method of erection used in balanced cantilever construction incorporates the use of a launching gantry, or an overhead truss, that is used to place the segments. Once a cantilever has been completed, the gantry has the capability to launch itself forward to the ahead pier to begin the erection of the next cantilever. One major benefit of this method is that it can be carried out entirely from above with no ground access required. This feature alone makes the launching gantry an excellent choice for bridges that have to be built in difficult terrain.

Launching Gantry Configuration

Launching gantries typically have one of two configurations. The gantry shown in Figure 1 has a single horizontal steel truss that is supported by cable stays. The cables transfer load from either side of the main truss to a center tower. The center tower is supported on the bridge deck. A mobile rear leg is also used to support the truss. A front leg is used to support the gantry during the launching and placing of pier segments. The front and center legs are fixed in position relative to the horizontal girder. The rear leg can move along the length of the girder. A lifting trolley rides along the bottom chord of the truss. Segments are lifted at the rear of the truss and delivered into position.

The gantry shown in Figure 2 has three support legs: a front, rear and center. The legs support the horizontal truss. The front leg may or may not be fixed in position, while the center leg and rear leg are free to move relative to the space truss. Segments are delivered to the rear of the truss. The lifting trolley that is used to place segments rides along the chord of the truss.

History of the Erection Method

The use of a launching gantry to erect a segmental bridge is not a new concept. The first bridge to be built employing a launching gantry was the Oleron Viaduct in France. The bridge connects mainland France with the island of Oleron, which is situated off the west coast roughly 80 miles north of Bordeaux. Erection of this 9,390-foot structure began in August 1964 and took 19 months to complete.

The structure is situated entirely over water. Traditionally, this type of structure would consist of concrete arches or steel structures, built using floating barges and cranes. However, due to wide tidal ranges in that area, the use of floating equipment would have substantially increased the time of erection and added to the project's cost.

In addition to these considerations, the erection of the bridge from the water and the low-lying marshes would have had a significantly adverse environmental impact on the nearby fragile Marennes oyster beds. The use of overhead erection equipment allowed this viaduct

to be built with minimal environmental intrusion.

The first bridge to be built in the United States that utilized an overhead launching gantry was over the Kishwaukee River near Rockford, Illinois. It was erected between 1978 and 1979. The bridge's twin structures provide a supplemental river crossing parallel to U.S. Highway 51. The total length of the bridge is 1,090 feet. It is composed of two 170-foot spans and three 250-foot spans. In order to protect the environmentally sensitive river valley, the design specifications required that the structure be erected completely from above. During the preliminary design phase, both steel and concrete structures were considered. The concrete alternative proved to be the most economically feasible solution.

The Hanging Lake Viaduct

The Hanging Lake Viaduct serves as an excellent example of a structure built over difficult terrain. This precast segmental bridge was erected in a narrow canyon that accommodates the Colorado River as well as two lanes of traffic on U. S. Highway 6. The Hanging Lake Viaduct completes the last remaining link of Interstate 70 in western Colorado.

After years of study, it was decided to construct I-70 through the scenic and environmentally sensitive Glenwood Canyon. Pressure from the public required that the design of the highway through the canyon disturb the natural terrain of the canyon as little as possible. In addition, there were no alternate routes available for detouring the traffic from I-70 at either entrance to the canyon.

Therefore, the Colorado Department of Transportation (CDOT) made it a design requirement that the Hanging Lake Viaduct superstructure be constructed from above. This mandate prohibited the use of any conventional ground-based equipment, falsework or temporary bents during construction that would adversely affect traffic and the environment.

Two complete designs were prepared to meet these unusually stringent but necessary restrictions. A precast concrete segmental box girder alternate was designed as well as a steel box girder alternate. The concrete alternate received the lowest bid at $34.1 million and was selected. Construction began in late 1989. The total project included 8,400 lineal feet of bridge. The typical span length was 200 feet, with parallel 300-foot spans over the Colorado River. The majority of the structure followed the river alignment on steep, rocky talus slopes.

The precast concrete segmental alternate was designed to be built using the balanced cantilever method with an overhead launching gantry. The contractor decided to use a gantry that had recently been used to build the French Creek Viaduct, located adjacent to Hanging Lake and separated from it by only a short tunnel. The gantry that was used on the project had been used to build several structures in Europe. The gantry was brought to the U.S. specifically to build the French Creek Viaduct.

The gantry is referred to as a "60/60 gantry." This designation is given to indicate that the gantry can be used to erect 60-meter (197-foot) spans and that it has a lifting capacity of 60 metric tons (132 kips). The gantry used on the project is similar in configuration to the gantry shown in Figure 1.

Special consideration was made during the bridge design to account for the erection method. Temporary cantilever post-tensioning tendons were added over the pier as well as negative moment post-tensioning at the midspan closure joints. This additional post-tensioning was added to accommodate launching the gantry onto the completed cantilever. In addition, a 60-ton counterweight was placed over the closure pour during launching.

The design specified that construction begin at a point where access by truck could be facilitated. This enabled all segments to be trucked to the gantry over the completed section of the new structure. Again, this eliminated the need for any ground-based cranes and kept the construction traffic off the sensitive terrain below.

Typical Erection Cycle. During a typical erection cycle, precast segments were symmetrically placed on either side of the pier using the gantry. The segments were then transported over the completed structure to the rear of the gantry. After that, the lifting trolley was used to place each segment in its final position. Segment weights varied from 40 to 55 tons and lengths ranged from five to eight feet. After

each segment was placed, six 1⅜-inch post-tensioning bars were stressed. The post-tensioning bars were used to suspend the segment while a matching segment was placed in the opposite position on the cantilever. After two pairs of segments were hung, a pair of tendons (12 by 0.6 inch or 19 by 0.6 inch), one located above each web, were stressed.

Once the cantilever was completed, closure beams were placed between the down station end and the previously completed structure, and the cantilever was adjusted for line and grade. A one-foot closure segment was then poured, making the just-completed cantilever continuous with the finished structure. Once sufficient strength was obtained (3,000 pounds/square inch) in the closure, the bottom slab continuity tendons were stressed. After closure, the gantry was launched ahead to the next pier. Prior to moving the gantry, temporary post-tensioning tendons were stressed.

The launch sequence is illustrated in Figure 3. Figure 3a shows the gantry position at the completion of cantilever erection. The rear leg has been moved forward as the gantry is being supported by the center leg and the stanchion. While the gantry is supported on the stanchion and the rear leg, it is launched ahead so that the center leg can be supported on the cantilever and the front leg can be supported on a steel pier bracket that is attached to the ahead pier (see Figure 3b). The pier bracket also resists the overturning moment during cantilever erection.

Next, the stanchions are retracted and the gantry is supported by the rear, center and the front legs as shown in Figure 3c. Once the gantry is in this position, it is used to place the pier segment. The pier segment is supported on temporary shims on the pier bracket. To complete the launching sequence, the gantry is supported on its front and rear legs, and the horizontal truss is launched ahead until the center leg is positioned over the pier as shown in Figure 3d. The gantry is lowered onto the center leg and stabilized by the rear leg.

The gantry was designed to build span lengths of approximately 200 feet. To enable the gantry to build the 300-foot spans over the Colorado River, a temporary bent was placed in the river. First, the maximum cantilever length possible was constructed with the gantry center leg in its typical position over the pier. Next, a support was installed under the leading edge of the cantilever. The gantry was then launched ahead so that the center leg was in position over the temporary support (see Figure 4). Cantilever construction could then continue for an additional 60 feet until the gantry could be launched to the next permanent pier.

The typical erection and launching cycles took anywhere from one to two weeks depending on the number of segments in the cantilever and the number of tendons to be stressed. The contractor was able to place approximately four segments (two pairs) during an eight-hour day. A typical cantilever contains 13 pairs of segments; therefore, segment placement took approximately six eight-hour days. The contractor was able to complete one 200-foot span every seven days over the last half of the project. This rate exceeded the contractor's own predictions and allowed the project to be completed four months ahead of schedule.

At the completion of erection, the gantry was disassembled and shipped to a storage facility for use on another job in the near future.

The New Baldwin Bridge

The new Baldwin Bridge was the first concrete segmental bridge to be built by the Connecticut Department of Transportation. It carries eight lanes of traffic across the Connecticut River between the towns of Old Saybrook and Old Lyme on the heavily traveled Interstate 95.

The new Baldwin Bridge's twin structures are 2,528 feet long and consist of eleven spans ranging in length from 177 to 275 feet. Its 488 precast trapezoidal box segments have a constant depth of 12 feet and a typical length of ten feet and seven inches. The eastbound structure has a deck width of 74 feet and eight inches, and the westbound structure has a deck width of 83 feet and nine inches. Segment weights range from 140 to 160 tons.

The original contract plans specified an erection technique employing two beam and winch setups to place the precast segments using the balanced cantilever method. Segments would have been transported directly below their intended erected location and hoisted into position. Segments would have been delivered by

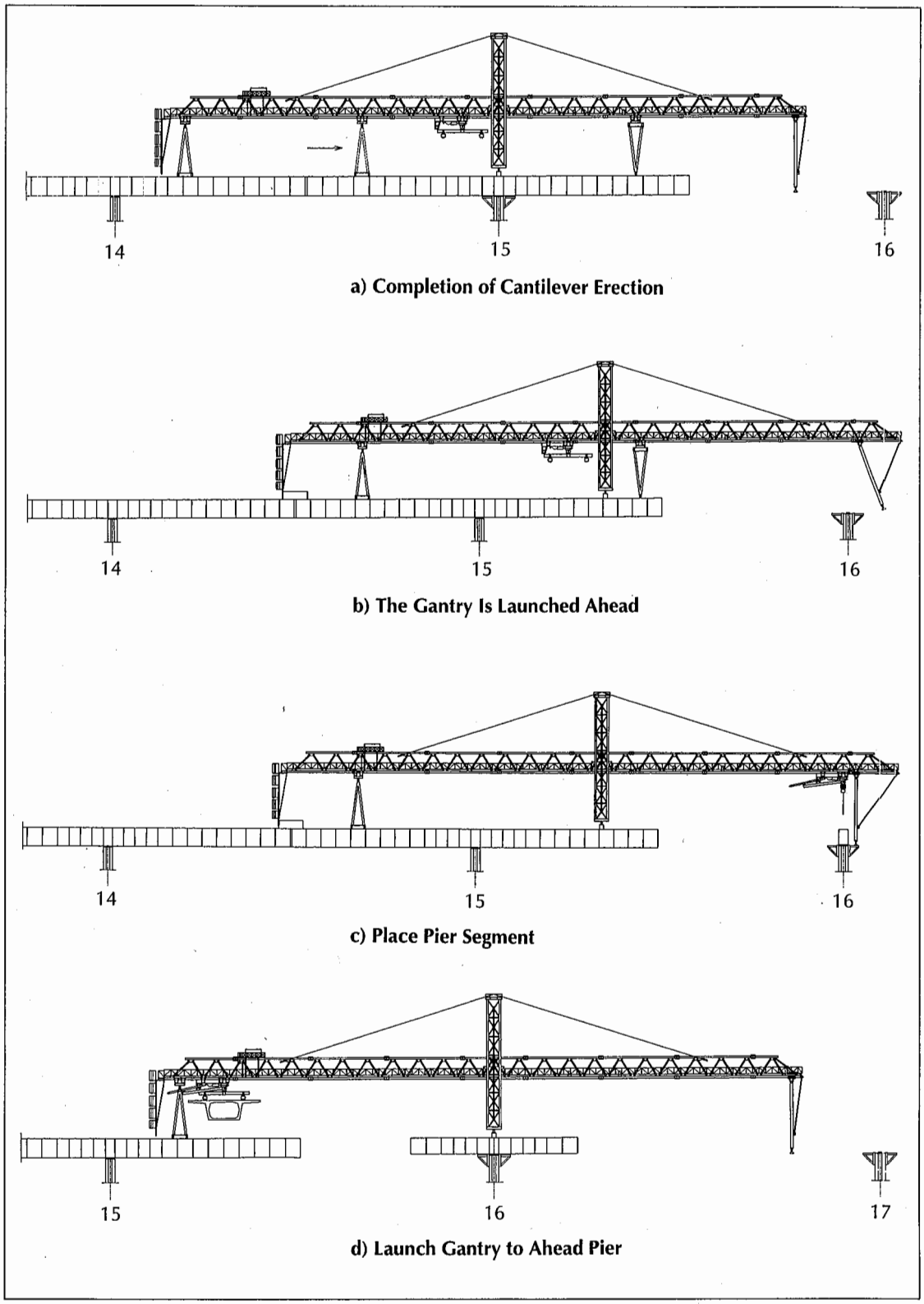


FIGURE 3. Hanging Lake Viaduct gantry launch sequence.

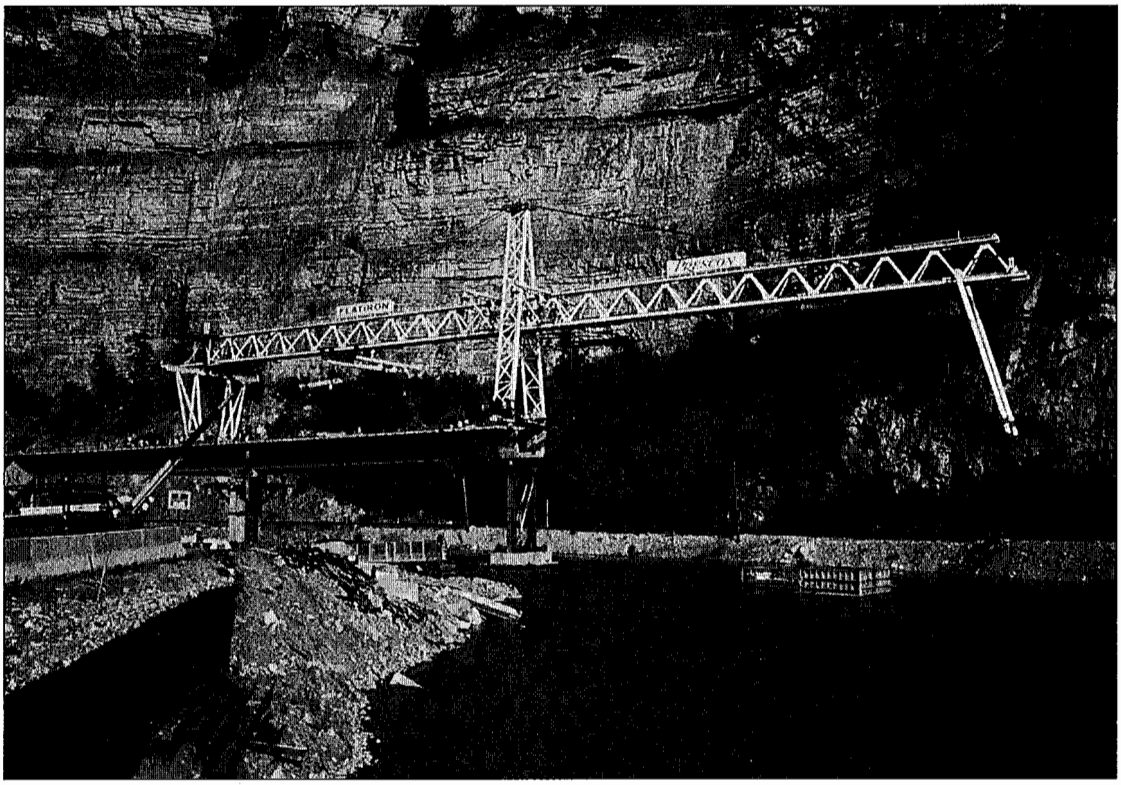


FIGURE 4. Hanging Lake Viaduct gantry launch over the Colorado River.

barge to the spans over the Connecticut River and by truck for the end spans over land. The contractor was concerned about the large amount of waterborne equipment required for this scheme. These concerns were based on having to work in strong tidal currents, satisfying the strict conditions of the environmental permit, having to keep the navigation channel open and being prohibited from dredging between April 1 and November 30. The use of large falsework bents also was a concern, particularly because of the varying height of each pier.

Because of these constraints, the contractor opted to explore the use of an overhead gantry system. This system would allow the erection of the structure with virtually no waterborne equipment, resulting in a significant time savings over the original scheme. The Baldwin Bridge gantry, shown in Figure 5, has an overall length of 435 feet and weighs approximately 600 tons (it is also similar to the gantry shown in Figure 2). It consists of twin triangular space trusses that have a depth of 17 feet. A twin truss design was chosen because of the structure's straight alignment as well as the weight of the segments.

The contractor wanted a piece of equipment with as much versatility as possible to allow for future use. The truss was supported on two legs, and moves with respect to these legs when launching. There was an auxiliary support located at the rear of the truss and a front leg at the nose of the truss. These supports were used to hold the truss up during the movement of the rear and center legs. The segment-lifting trolley rode on the top chord of the truss and employed a unique hydraulic jack with a 17-foot stroke and 200-ton capacity to hoist and position the segments. The overturning moment encountered during cantilever erection was resisted by the stabilizer arms that connected the deck to the truss as the cantilever was built. The stabilizer arms eliminated the need for falsework at the piers. Additional equipment attached to the gantry included two movable work platforms that were used for prestressing and an

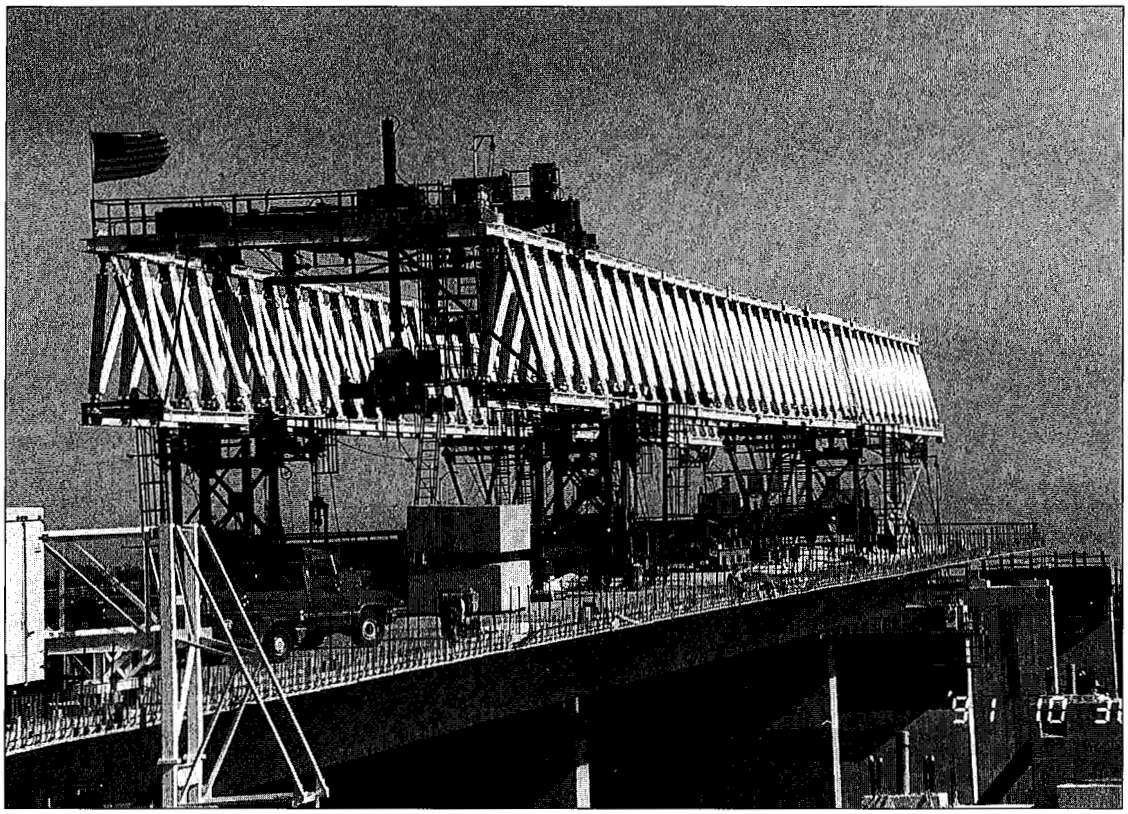


FIGURE 5. A view of the new Baldwin Bridge launching gantry.

electrical generator that was used to power the gantry.

The gantry was fabricated in Canada and trucked to the site in pieces. It was assembled and load tested on the western approach of the westbound structure. The truss was designed to be easily transportable with pinned connections between the chords and diagonals.

Erection began on the westbound side of the bridge. The gantry worked its way to the eastern side, using the already completed structure to provide access for the 12-axle segment hauler. Once the westbound structure had been completed, the entire gantry was loaded onto two special hauling rigs and moved back over the completed bridge into position to repeat the process for the eastbound structure. After the eastbound structure had been completed, the gantry was disassembled and trucked to storage to await its next assignment.

The bridge was designed to resist the loads of a comparatively small beam and winch placed at the ends of the cantilever. Though the loads from the erection gantry were much greater, the structure was able to resist the weight of the gantry with minimal modifications. The most significant of these modifications were the addition of external post-tensioning and a counter-weight over the middle of the previously erected span used during launching.

Typical Erection Cycle. The gantry was launched ahead so that the front leg was resting on the ahead pier. Jacks under the front leg were activated and the segments directly over the pier were erected as shown in Figure 6. The center leg was then moved to the pier and the rear leg to the end of the previous cantilever. This alignment allowed the truss to be launched ahead into the cantilever erection position. As the segments were erected, the stabilization arms were attached to the deck in order to resist overturning moments. Two stabilizer arms were required during this operation. The major stabilizer was used to resist any unbalanced moment, and the minor stabilizer was

used only when the cantilever was in a balanced position and the major stabilizer was being moved ahead. A counterweight was also used on the cantilever to reduce the unbalanced moment.

Each segment was epoxied to the previous segment. The epoxy was squeezed and the segment temporarily suspended using post-tensioning bars that were coupled at the face of the previous segment. After each pair of segments had been erected, several post-tensioning tendons were stressed in the top slab. Once all segments were placed, a one- to three-foot closure joint was poured to join the cantilever and the previously erected span. Continuity tendons were stressed after the closure had reached sufficient strength. The gantry was then launched ahead to begin the erection of the next cantilever. A typical cantilever required two to three weeks to erect.

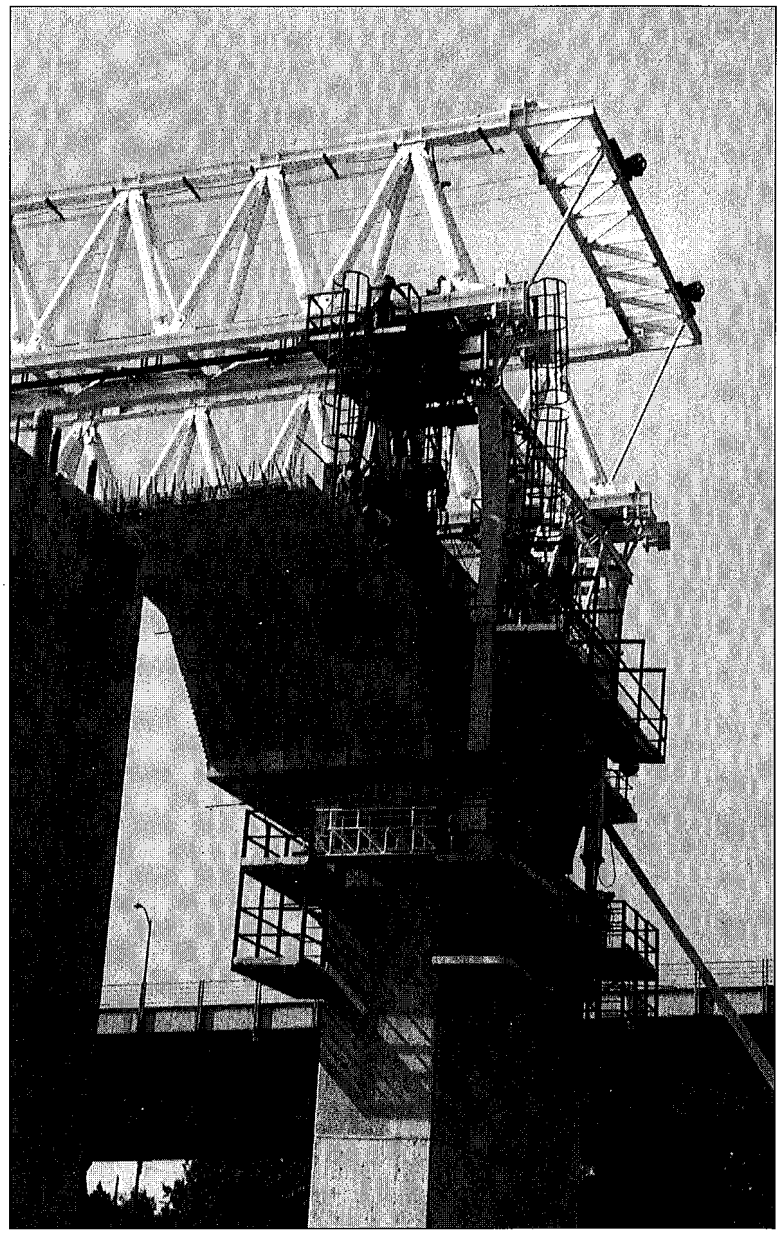
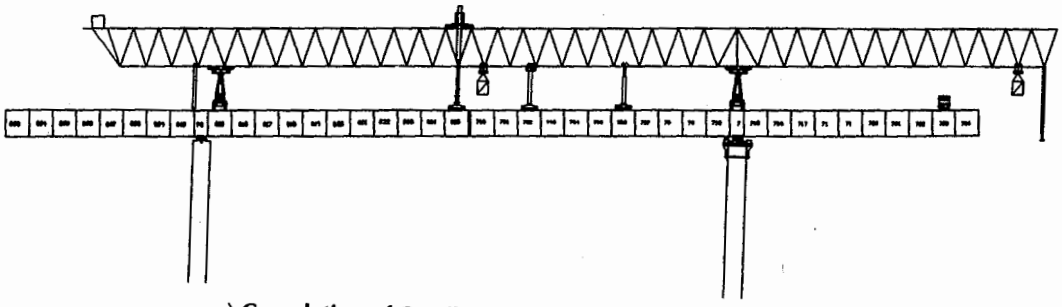


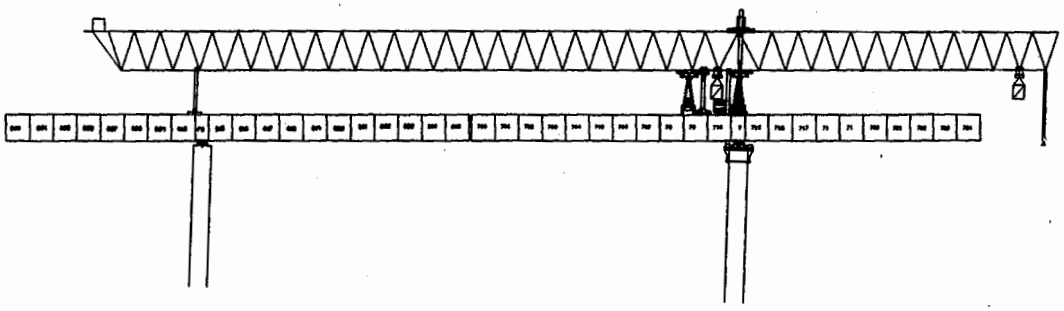
FIGURE 6. Pier head erection on the new Baldwin Bridge.

After a cantilever was completed and the back closure had been made, the gantry was positioned as depicted in Figure 7a. Figure 7b shows the gantry supported on the center leg and the auxiliary support. The rear leg is moved ahead. In Figure 7c and 7d the gantry is supported on the rear or center leg and the auxiliary support. The rear and center legs are repositioned in preparation for launching the gantry. Figure 7e shows the launching of the gantry to the ahead pier. In Figure 7f the gantry is supported on the auxiliary support, the front leg and the center leg. The rear leg is repositioned and two segments are placed over the ahead pier. Figure 7g shows the center leg being moved to the ahead pier where it is supported on the pier segment. The final launch step is illustrated in Figure 7h. With the gantry supported on the rear and center legs, the truss is rolled ahead into the pier erection position.

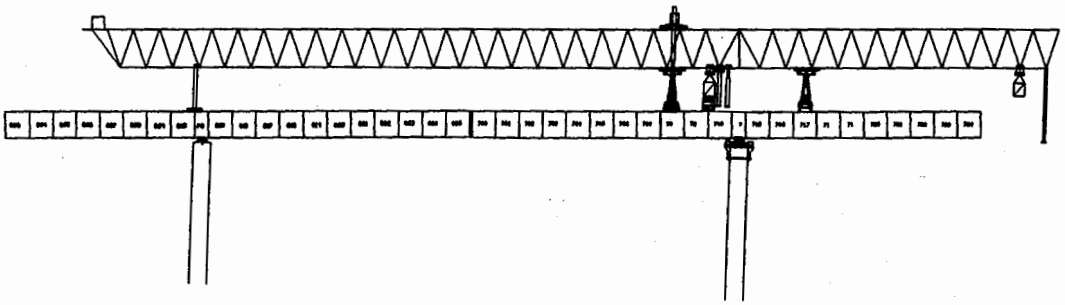
The new Baldwin Bridge serves as an excellent example of a structure built in difficult terrain where an overhead launching gantry was used. The erection of the new bridge was



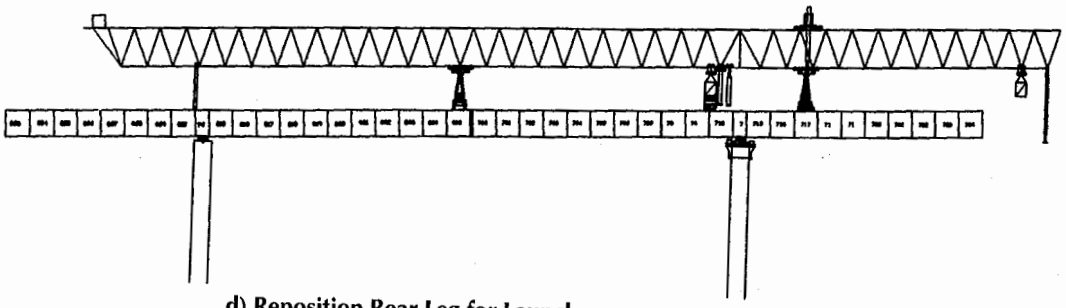
a) Completion of Cantilever Erection



b) Reposition Rear Leg So That Center Leg Can Be Moved Ahead



c) Reposition Center Leg



d) Reposition Rear Leg for Launch

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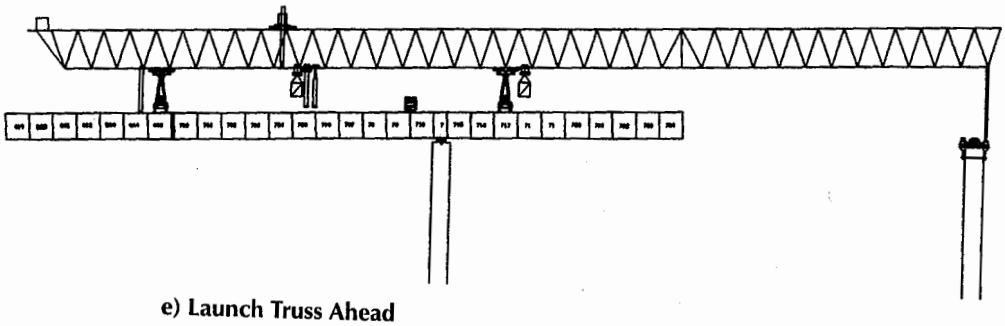
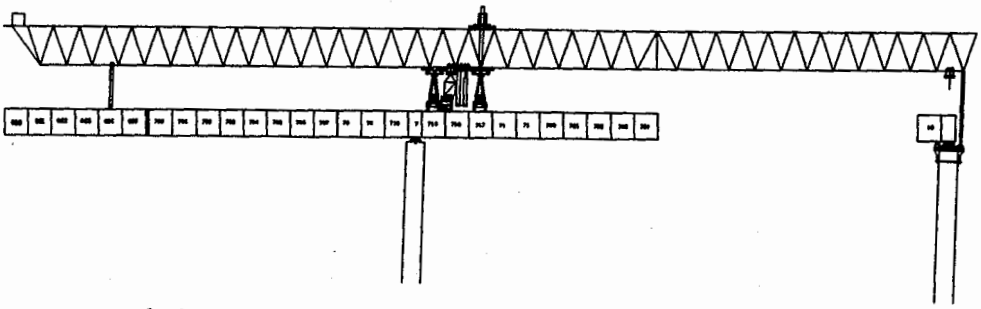
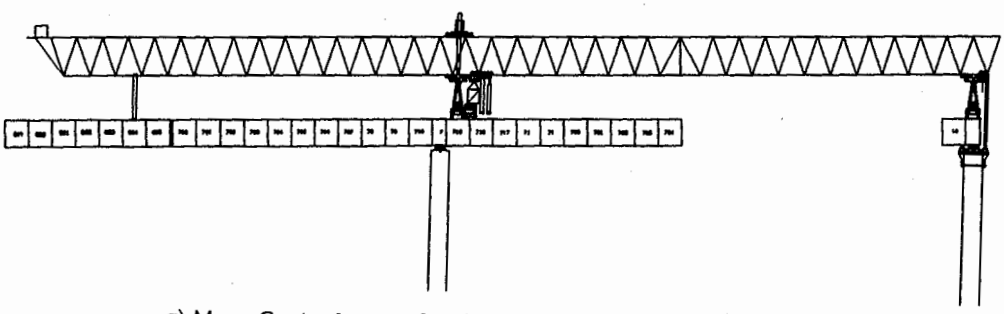
FIGURE 7. The new Baldwin Bridge gantry launching sequence.



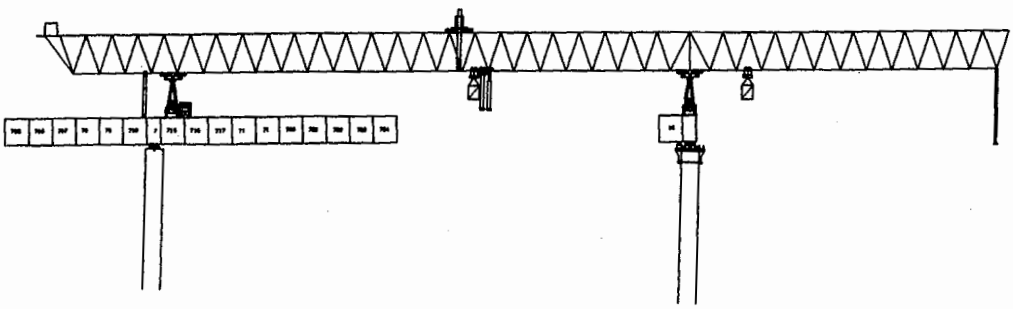
e) Launch Truss Ahead



f) Place Pier Segment



g) Move Center Leg to Ahead Pier



h) Launch Truss Ahead Into Cantilever Erection Position

completed in the spring of 1993, six months ahead of the contracted schedule. The contractor was able to surpass the contract schedule in large part due to the use of the overhead launching gantry. The up-front investment required for this specialized piece of equipment resulted in time and cost savings in the end. This gantry can be modified to meet many different bridge configurations and should be considered for use in any situation where site constraints limit access from the ground.

Conclusion

As more and more bridges are built over increasingly difficult terrain and under strict environmental constraints, the idea of building entirely from above becomes more of a necessity than a luxury. The overhead launching gantry allows concrete segmental bridges to be erected entirely from above, which is a necessity given certain site parameters. The two case studies presented here — the new Baldwin Bridge and the Hanging Lake Viaduct — are both excellent examples of the use of this erection method. They demonstrate how the use of an overhead launching gantry can save time and, therefore, money on projects built in difficult terrain.

ACKOWLEDGMENTS — *The Flatiron/Prescon Joint Venture was the contractor for the Hanging Lake Viaduct project. The gantry for that project is owned by Prescon Corp. and was used by Campenon Bernard in Europe. The contract for the new Baldwin Bridge project was the joint venture of Perini, PCL and O&G.*

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Design for Tunnel Safety: I-90 Tunnels, Seattle

Safety issues, including the transport of hazardous cargoes, were the primary focus of the design of the mechanical and electrical systems for two highway tunnels.

PHILIP E. EGILSRUD & GARY W. KILE

In the early 1960s, the Washington State Department of Transportation (WSDOT) began construction of the final seven-mile segment of Interstate Highway 90 that runs from Boston to downtown Seattle. The original plan included: a 14-lane freeway crossing Mercer Island with a deep cut through Mercer Island's First Hill; a new wider floating bridge (adjacent to the existing bridge which would be rehabilitated); and an open cut or multiple tunnels through Mount Baker Ridge on the Seattle side of Lake Washington.

However, in February 1971, during the preliminary stages of construction, a well organized anti-highway group obtained a "stop construction" injunction in Federal Court. The primary basis of the injunction was that the project's Environmental Impact Statement (EIS) did not adequately address the environmental impacts. Following more than eight years of environmental studies, public meetings, development of alternative concepts, and negotiations, the project's Final EIS received approval and the injunction was lifted in August 1979.

The final settlement resulted in a significantly altered design that included the use of "lidded" cut-and-cover and bored tunnels to mitigate impacts to area communities and aesthetically preserve the scenic hills. A plan for the two highway tunnels was developed for the final seven-mile segment replacing U.S. Route 10 that crossed Lake Washington and Mercer Island, and included the famous Mercer Island floating bridge crossing the lake (see Figure 1). On the west side of the lake, a pair of two-lane tunnels carried U.S. Route 10 through Mount Baker Ridge, a ridge 200 feet high along the west shore of the lake. Preliminary design started in 1982 and the tunnels were completed in 1989.

Tunnel Descriptions

The First Hill Mercer Island lid is a 2,850-foot long tunnel, consisting of three side-by-side, cut-and-cover constructed cells (see Figure 2). The tunnel has a "humped" vertical profile (see Figure 3) and its roadways are constantly curving in order to go around the foot of First Hill.

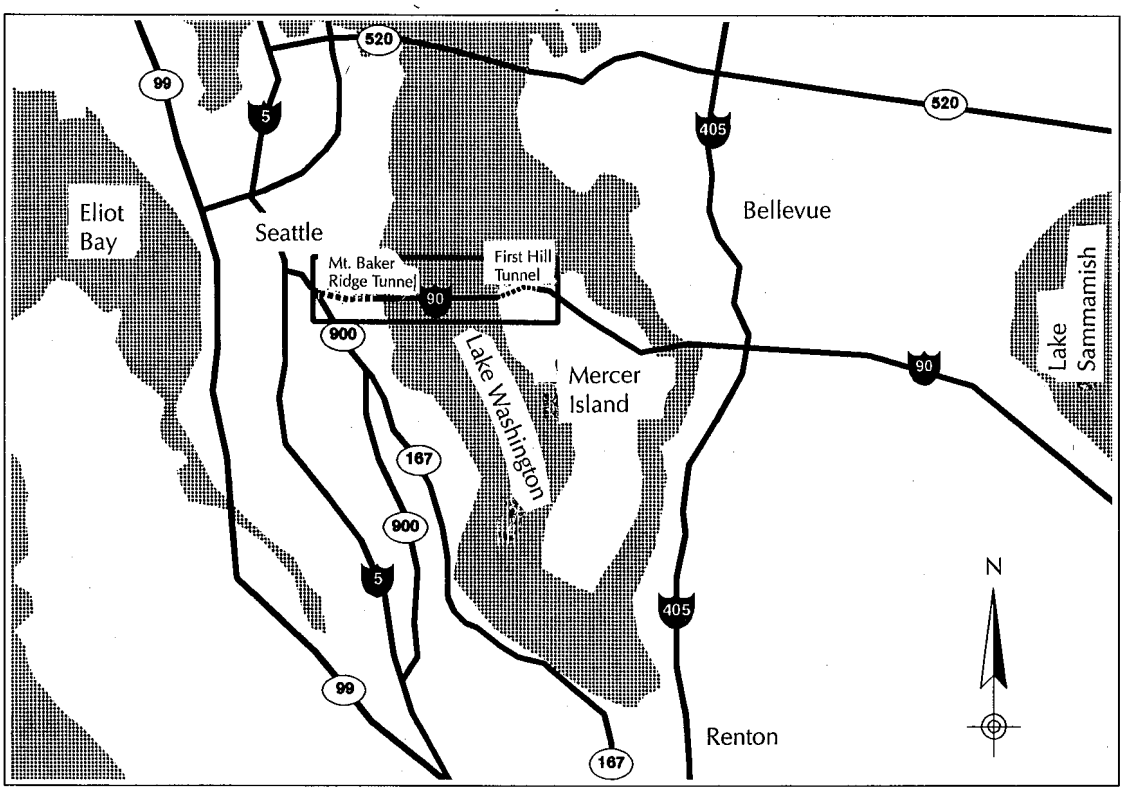

FIGURE 1. Location of the tunnel projects.

The tunnel arrangement provided considerable space above the 18-foot tunnel ceiling for the placement of supply and exhaust ducts and fan rooms. A park was constructed on the lid surface.

The Mount Baker Ridge tunnel/lid is a 3,400-foot long tunnel. It has three roadways—translating into three separate cells in the tunnel. Its westbound and center high occupancy vehicle (HOV) lanes were built while all traffic ran on the existing U.S. Route 10 tunnel roadway. After those lanes were completed, all westbound traffic used the final westbound lanes and eastbound traffic used the center

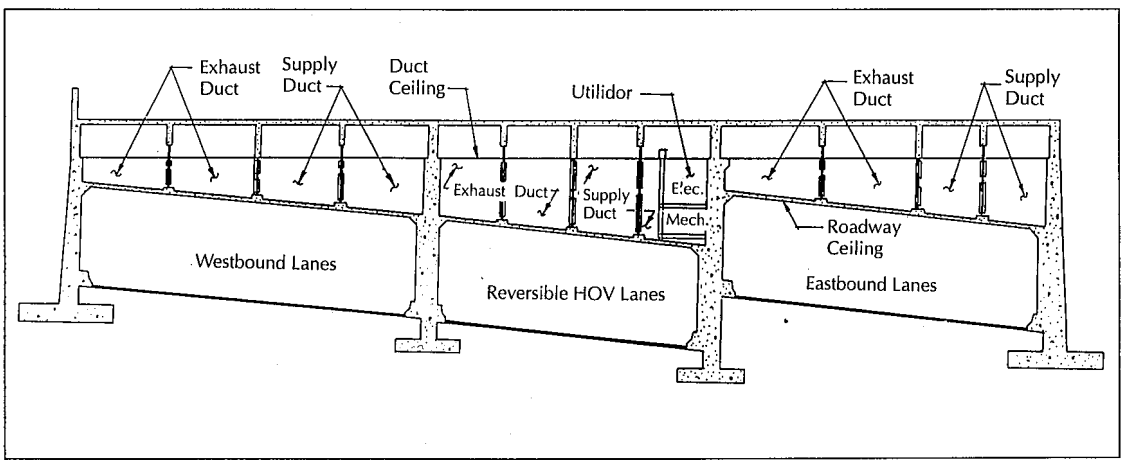

FIGURE 2. Typical section of the First Hill Mercer Island tunnel.

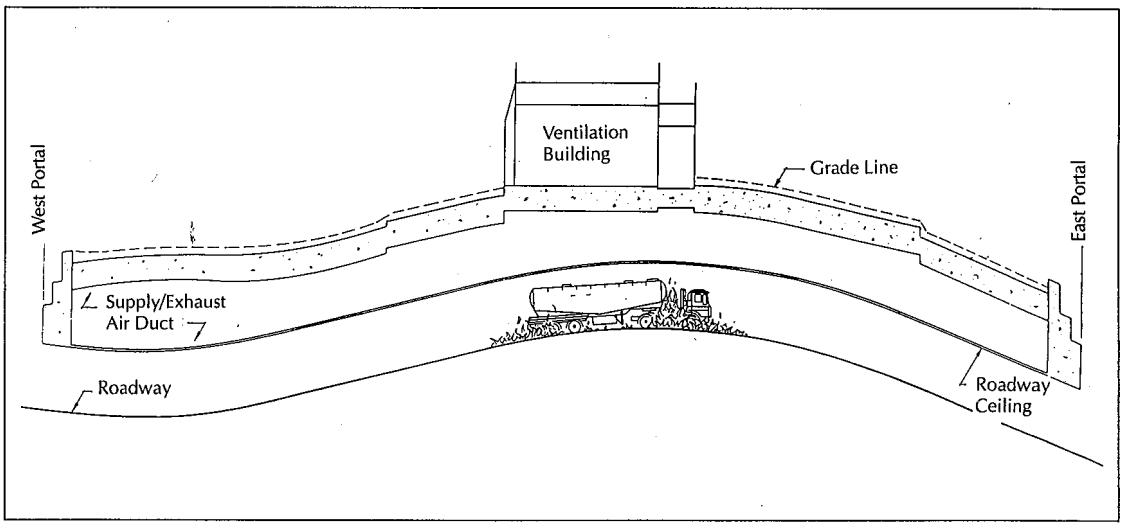

FIGURE 3. Typical profile of the First Hill Mercer Island tunnel.

roadway that was striped for three lanes. The old U.S. Route 10 roadway was left free for reconstruction into the final eastbound roadway.

Lengthwise, the Mount Baker Ridge tunnel/lid is composed of two segments. The 1,400-foot long east segment contains the westbound and center HOV roadways in a large 63-foot (inside diameter) bore, stacked with the westbound lanes in the middle, the center HOV lanes below, and a bicycle/pedestrian path above (see Figure 4). The eastbound lanes are placed in the renovated twin bore tunnels for U.S. Route 10, two lanes in one and one lane in the other.

The west segment is about 2,000 feet long and has three cut-and-cover cells, side by side, each with air ducts behind the side walls. The center HOV (going east) lanes, at about 1,000 feet into the tunnel, begin to drop and curve under the westbound lanes to enter the large bore segment.

Safety Issues

Principle safety issues for the tunnels were as follows:

- Being able to handle any incidents involving vehicles carrying hazardous cargoes that are allowed to transit both tunnels;
- Conforming to local fire department requirements for tunnel fire protection;
- Providing redundant or complementary systems for incident detection;
- Ensuring adequate and reliable fire hose valves and water supplies;
- Ensuring a continuous supply of electrical power;
- Providing access by towing and emergency vehicles that is complicated by the location of interchanges, and the roadway and bridge geometry of the highway before and after the tunnels;
- Reducing or eliminating problems regarding an east/west orientation and the "black hole" effect of entering the tunnel against a low sun and the reflection of light off the neighboring lake in the morning and evening periods;
- Providing escape exits during fire and spill emergencies or other incidents; and,
- Providing traffic control of high-speed traffic during emergency incidents in the tunnels.

Transporting hazardous cargoes, including 10,000-gallon gasoline tankers, could not be prohibited in the tunnels because of political and economic reasons as well as the lack of any satisfactory alternate routes.

The WSDOT decided that the Seattle and Mercer Island fire departments would be responsible for responding to fire emergencies in both tunnels.

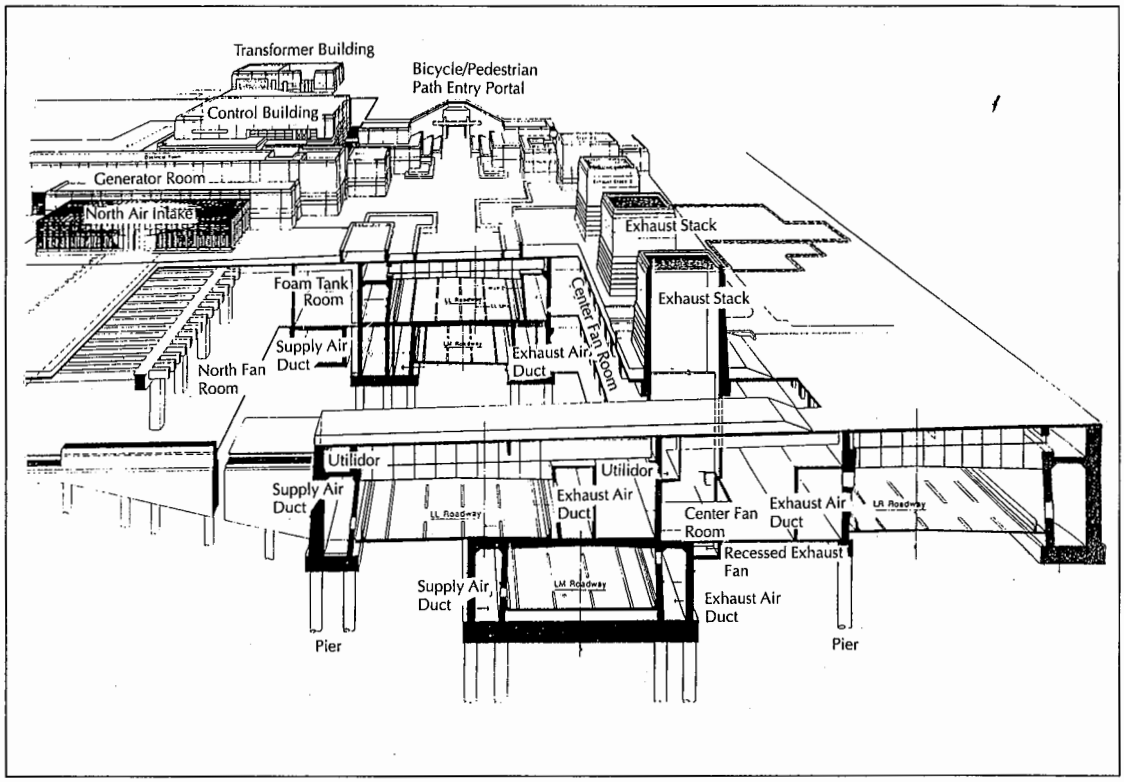

FIGURE 4. Cut-away view of the Mount Baker Ridge tunnel and associated structures.

Safety Philosophy

Safety was a paramount concern in these tunnels. The design effort focused on a two-fold approach: the creation of safety systems for incident prevention and management; and the creation of control systems for response to, as well as detection and control of, incidents. Included as a significant factor in the design of the safety systems were the safety aspects of the roadway geometry for both tunnels.

Safety Systems

The following systems were developed for incident prevention and management.

Fire Protection. The Seattle and Mercer Island fire departments required that foam sprinkler systems be installed in each tunnel where vehicles carrying hazardous cargoes would be permitted. Design concerns for the fire protection system were as follows:

- Location of sprinkler heads;
- Required foam capacity;
- Available water supply/required water capacity and pressure;
- Freeze protection of sprinkler system;
- Most effective sprinkler system for foam/water application;
- Accidental or normal foam discharge on moving traffic;
- Corrosion protection of sprinkler system; and,
- Storage and distribution of foam concentrate.

Studies using applicable computer modeling for different fire schemes (size and location of the fire in the tunnel) determined that the center HOV cells would not require sprinklers. This decision did not have an impact on normal operation; however, during the construction period when the eastbound traffic used the center HOV roadway temporarily, hazardous cargoes had to be banned.

Reasoning that discharges from sprinklers installed on walls would be blocked by large trailer trucks in the close-by lanes, it was de-

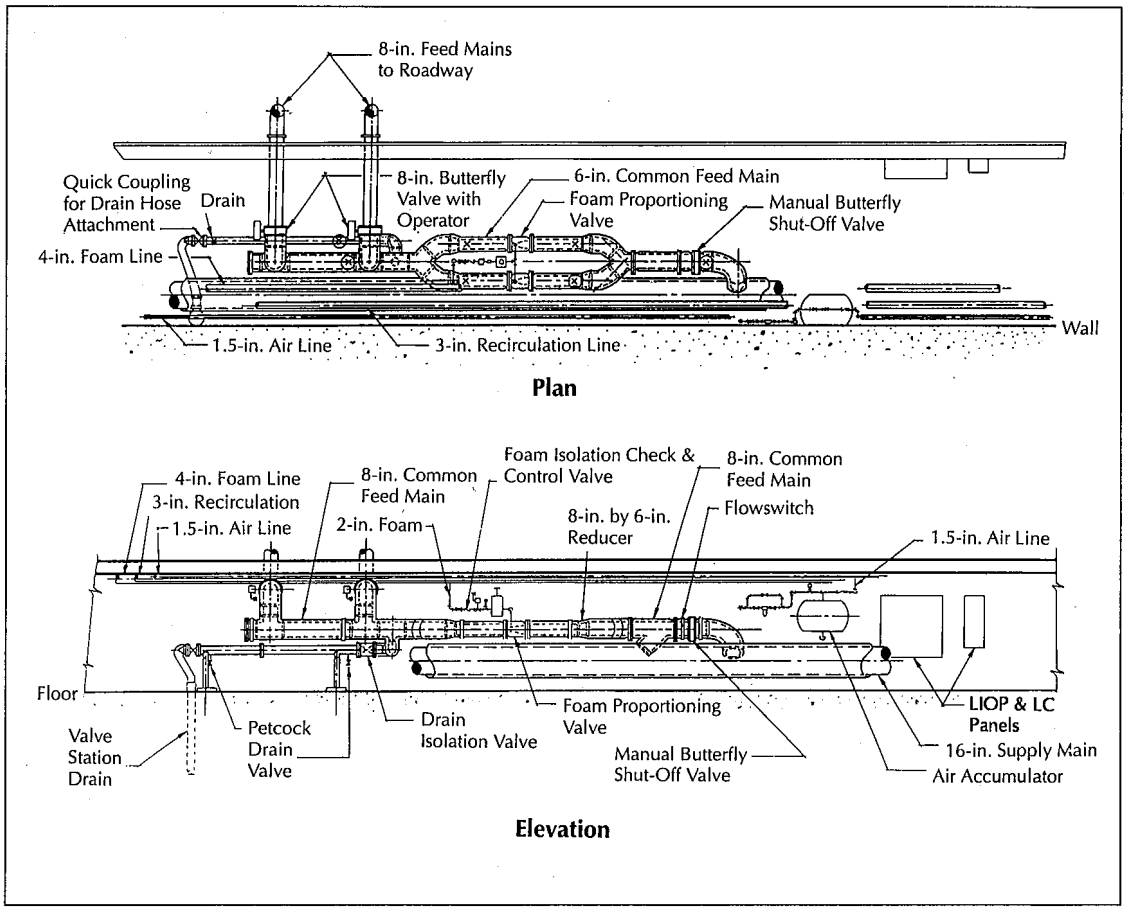

FIGURE 5. Piping arrangement for the valve stations.

cided to install sprinkler heads on the ceiling spraying downward.

To meet National Fire Protection Association (NFPA) requirements, a water flow rate of 0.16 gallons per minute (gpm) per square foot with a foam concentrate of three percent was the basis of design.

The open deluge-type sprinkler system is divided into zone arrays approximately 150 feet long (along the tunnel ceiling), spanning the width of the roadway. Open sprinkler heads spaced on a ten-foot by ten-foot grid are fed by a piping array that was designed to be pressure balanced for the particular zone. The array piping was designed for freeze protection by allowing the piping to drain through the heads after flow is stopped.

A valve station located near each zone array provides the water/foam proportion control for each zone (see Figure 5). Foam concentrate is added at this valve station through a proportioner set to provide the desired water/foam mixture percentage.

This design allows two sprinkler zones to be activated at one time without exceeding the system's water capacity from municipal water supply mains. If a fire is located in a border area between two sprinkler zones, both zones can be activated without overloading water capacity.

Each tunnel was equipped with a foam concentrate storage and pumping center that pumps the concentrate to each valve station. All control valves on both the water supply and the concentrate piping are pneumatically operated.

Fire Hydrant & Hose Valve Systems. All tunnel cells are equipped with a fire main with hydrants. Since all tunnel roadways are built on the earth (except in the Mount Baker Ridge large bore segment), wet water mains were buried under the pavement with regular-type

hydrants in covered pits in the shoulder areas. Spacing, hydrants and capacities were designed in accordance with recommendations in NFPA 502.

Tunnel Roadway Drainage. Both tunnels have "humped" profiles with roadways high at the midpoint and low at the portals. Roadway drains are piped to stormwater sewers that run into Lake Washington or other environmentally sensitive receptors.

Drainage from the tunnels could be contaminated by the following:

- From tunnel interior washing (detergent, oil, filth, etc.)
- Foam sprinkler dumps
- Spill of fuels or cargoes

To prevent any of the above from flowing into the lake, at each portal the outfall drains have manually-operated valves that divert contaminated flow into holding tanks from which the contaminated water can be trucked away for proper disposal.

Ventilation. The EIS mandated a fully transverse ventilation system for the tunnels. Each roadway of each tunnel was equipped with three supply and three exhaust fans, except for the Mount Baker Ridge eastbound lanes which have six exhaust and three supply fans (totaling 39 fans for the two tunnels). Fan rooms were located at the center of each tunnel. Ducts split and parallel the roadways to each portal so that air is evenly supplied and exhausted along the total length of each roadway.

The selected intake fans are centrifugal double-width, double-inlet-type with a single motor controlled by variable frequency drives (VFDs) providing fan speeds that vary from ten percent to full speed. Exhaust fans are ducted type with inlet boxes. Fan housings are fabricated of stainless steel for anti-corrosion protection, thereby eliminating the need for constant painting. The exhaust fans are sized much larger than the supply fans to help clear the tunnel of smoke due to a fire and exhaust fumes during periods of heavy traffic.

Emergency Cabinets — "SOS Boxes." Wall niches with an emergency telephone, fire alarm pull boxes and a 20-pound dry powder type fire extinguisher were installed every 350 feet on both walls of all roadway cells, opposite one another to prevent people from crossing lanes to access the nearest cabinet. These niches were marked "SOS" and painted a distinctive color.

Communications. In the event of an incident, communications regarding the incident are of great importance. The First Hill lid and the Mount Baker Ridge tunnel/lid were equipped with such communication systems as:

- Radio rebroadcast antennas, spanning the full length of the tunnels, to allow emergency services personnel to communicate with their dispatchers; and,
- A commercial radio rebroadcast system with an override system to advise vehicle occupants of critical situations in the tunnels.

Tunnel Lighting. Lighting of a tunnel, particularly during daylight hours is an important factor in overall tunnel safety. Over the years many opinions have been expressed, papers written, and designs made for solving the problem of a motorist entering the darker interior of a tunnel from bright sunlight. Of particular concern was that motorists driving west on the floating bridge on Lake Washington approaching the Mount Baker Ridge tunnel portal would have problems with adjusting to differences in lighting because the direct afternoon sunlight from low over the Mount Baker Ridge and light reflecting off the lake shines in motorists' eyes.

A "counterbeam lighting" system was recommended for both tunnels. Counterbeam high pressure lighting employs point source controlled lighting where, in the threshold and transition zone, the luminaire directs a beam of light at a 45-degree angle towards the driver. The pavement appears bright due to its specular characteristics, thus improving contrast and visibility for motorists entering the tunnel. This method of controlling the luminaires' light distribution to increase contrast allowed a reduction in the overall lighting power consumption, initial installation and maintenance costs.

Escape Means. Both tunnels were equipped with escape exits to the surface. These exits are located in the center of each tunnel, which all roadway cells could access through cross passage.

Ease of Maintenance. The tunnel systems were designed for uncomplicated maintenance. Equipment and materials such as electrical boxes, wireways, fan housings, tunnel lighting housings and supports were made of stainless steel to decrease the need for maintenance such as painting and repair due to corrosion. Structural steel was also protected from corrosion for long-term life. To prevent corrosion in the extensive piping system, welded joint stainless steel tubing was used for all sprinkler and foam concentrate piping. Victaulic couplings and expansion joints were used throughout the water/foam concentrate distribution piping system to provide flexibility and ease of maintenance.

The structure itself was so designed that any part of the structure could be accessible for inspection via access passages in case of an earthquake. Also, any structural steel that could be exposed to excessive heat was fireproofed.

Tunnel luminaires were a custom design. Housings were designed to allow maintenance personnel to remove and replace an entire luminaire without any special tools.

Safety Aspects of Roadway Geometry

During the early design phase of the project, there was concern about the geometry of the roadways of the First Hill facility and the center roadway of the Mount Baker Ridge tunnel. As noted before, the First Hill roadways were "humped," rising in elevation to a high point at the center and downgrade to the exit portal (either way of traffic direction). At the same time, these roadways curved and were superelevated accordingly. Further complicating matters was the project architects' requirement that the side walls be tilted inward, narrower at the ceiling and wider at the roadway level.

However, vertical and horizontal curvatures were gentle enough that neither the walls nor the ceiling obstructed the driver's view of the roadway.

Well aware of the importance of tunnel roadway geometry in tunnel safety, the project designer and WSDOT carefully studied this aspect of the design of these tunnels. For the First Hill lid, the concern was the fact that neither the roadway, ceiling or walls related to the horizontal or vertical and that these visual parameters continually change as the driver proceeds through the tunnel. At First Hill, the changes are in one direction horizontally and the curvature does not reverse. Vertically, the ride is first up and then down. It was decided, after some discussion, to keep the ceiling parallel to the roadway.

The center roadway reversible HOV lanes of the Mount Baker Ridge tunnel going east drops and turns under the adjacent westbound roadway and then proceeds straight as an arrow to the east portal through the large bore segment. Realignment to the center position takes place on bridges over the lake outside the tunnel.

To evaluate these curving roadways with respect to possible driver disorientation and consequent possible safety problems, WSDOT employed a computer program that used perspective drawings from a driver's viewpoint. The drawings were then showed in rapid sequence to show how the driver's view would change as the vehicle proceeded through the tunnel. This effort convinced WSDOT that the geometry would not be a problem and would not cause driver disorientation or become a safety problem.

New Jersey safety barriers were installed against each interior tunnel wall. There were no provisions for a sidewalk, high or low, for fear that any activity by people on the sidewalk might distract motorists and cause safety problems. The 18-foot clearance allows 1.5 feet for tunnel lighting, signs, traffic lights, etc.

Tunnel Systems Control

The two tunnels have numerous systems, mostly related to safety and/or fire protection, that require a control mechanism for optimum operation. The size of these systems, as well as the amount of information coming from them, dictated that the control system be electronic based.

In addition, each tunnel needs to be controlled from a centralized location. WSDOT decided that around-the-clock staffing of a control room in each tunnel would be too costly. Instead, it was determined that both tunnels would be controlled from consoles located in a freeway control center called the Traffic Systems and Management Center (TSMC). Personnel at the TSMC could "double-up" and tend both tunnel control consoles, as well as the freeway controls.

The computer control system is interactive — each system to be controlled is translated into a schematic that includes real-time indicators of the system's current status. This information can be called up on the computer's video monitor by use of a menu. This diagram changes as the system responds to commands from the operator or other sensors.

There are many separate subsystems involved in controlling the Seattle and Mercer Island tunnels. These subsystems are designed to provide monitoring and control functions for tunnel lighting, traffic control, ventilation, communications and power distribution. Each subsystem is part of the total Supervisory Control and Data Acquisition (SCADA) system that controls the tunnel. This system uses modern techniques of distributed control to dramatically increase system reliability by eliminating system interruptions due to single point failures.

The control systems are electronic and operate using analog signals, discrete (voltage level) signals and digital signals. Analog signals are produced by sensors measuring light intensity, carbon monoxide, voltage, current, power, fan speed and various other parameters. Most of these data are available through the tunnel computer system.

Programmable Logic Controllers (PLCs). The SCADA system provides monitoring and control capabilities of the tunnel operating parameters. Analog and discrete values are first measured by a subsystem such as a programmable logic controller (PLC). The PLC passes information to and from the tunnel computer over a high-level communications network (digital signal), and from there information is transmitted to the TSMC through a fiber optics network.

A failure at the TSMC or in the fiber optics line will not require tunnel closure. Likewise, tunnel computer failure does not require tunnel closure because locally distributed PLC control systems can operate independently. If an individual PLC system or component should fail, a redundant or parallel PLC (operating parallel systems within the tunnel) continues to operate the tunnel in a safe and orderly manner. In the event of multiple subsystem control failures, critical items, such as lighting and ventilation, are manually controlled in the local equipment location.

The PLCs are stand-alone subsystems and normally perform control functions based on their inputs from field sensors. They also accept inputs from the tunnel computer system regarding new control parameters or setpoints. The PLCs communicate directly with each other as well as with the tunnel computer.

Due to the safety requirements of monitoring and controlling a highway tunnel, a highly reliable information acquisition and control system with backup capabilities is required. Various means are used to obtain the desired reliability, including redundancy of sensors, standby equipment, distributed control, etc. Each tunnel has a distributed control system with five PLCs. Functional control of similar systems such as lighting, traffic signal heads, and ventilation are divided among four of the PLCs. The fifth PLC has a redundant processor and controls the substations, switchgear and standby generators where control cannot be easily divided into two separate, independent systems.

Because of the substantial length of the tunnels, a distributed digital control system with a local input-output panel (LIOP) at each valve station was selected. Another LIOP in the foam room activates foam concentrate pumps and automatically operates recirculating freeze protection. All LIOPs are supervised by a central processing unit in the control console.

Tunnel Computer System. Each tunnel is equipped with a host mini-computer operating primarily on data supplied by the operator, the PLCs and other remote subsystems. It is used to supervise the operation of the PLCs and make setpoint control changes as determined by the operator. The computer system also handles data gathering and historical files, alarm/status logging and report generation.

Inputs to the tunnel computer include the PLCs, traffic data stations, fire system, essential services and inputs from the TSMC remote control system. All inputs and outputs are available for display on a color video monitor and are accessible by operator request.

Environmental Control Panel. A separate environmental control panel is located in the control room with limited monitoring and control capabilities for the tunnel. The panel has two purposes:

- To generate a status report of tunnel operating conditions for personnel — without requiring operational knowledge of the computer system; and,
- To provide control of critical functions in the event of system control failures or during maintenance work periods on various control equipment.

The environmental control panel alone is not intended for long-term control of the tunnel.

Fire Protection. Upon detection of a fire, the computer control system increases exhaust ventilation to 100 percent (100 cubic feet per minute per lane foot) and maintains supply at 20 percent in the effected vehicular zone. Adjacent zones operate normally. In addition, tunnel closed signs come on at each portal, traffic signals upstream go to red and downstream stay green, and the tunnel lights are set to maximum brightness. These responses are programmed; however, the control room operator can change them if desired by the local fire departments or others.

The activation of a sprinkler zone is accomplished through a computer-based control that automatically activates a particular zone upon signal from a ceiling-mounted fire detector located within that particular zone. To give the control room operator time to investigate the seriousness of the fire, traffic flow and other conditions, there is a 30 to 90 second delay after the alarm sounds before the automatic response is actuated. The operator may leave the system in the auto mode and let the countdown end at the initiation of foam. However, the system also allows manual selection of one of following three alternative options:

- Abort/Reset: Sets the countdown back to seconds or resets the foam system completely if the detectors are out of alarm.
- Dump: Immediately opens the valve in the first two zones that detected an alarm.
- Silence: Continues the countdown but shuts off the alarm horn.

A separate graphic display of the tunnel's fire situation with the manual option is also located in the control room. This independent manual panel with the ability to intervene in the automatic sequence also helps to manage another problem — a blinded operator that can occur during a serious fire. For example, the detectors may report and then melt for hundreds of feet. In auto mode, the system only dumps foam on the first two zones to report a fire. In the manual mode, the operator (or the fire department) can follow the fire as long as the foam and visual observation last or communication is maintained between the fire area and the control center.

The delay period with manual response override is important with regard to SOS callbox alarms. If the door on the fire extinguisher box is opened, a door switch sounds an alarm in the control room. The SOS box location is also indicated on the console, as is the video camera in which the particular SOS box appears. Emergency telephones automatically dial 911, the control room of that particular tunnel is notified, and the tunnel control room operator can listen in. The purpose of this alarm and indicator is to:

- Inform the operator that someone needs an extinguisher to put out a fire; or,
- Someone is stealing the extinguisher.

In the latter case, the operator can abort an obvious false alarm. In the former case, the operator can judge the level of response appropriate to the incident.

Ventilation Fans. Fan speeds are controlled by PLC logic and are normally operated on a time-of-day schedule based on traffic loads with an automatic speed increase if carbon monoxide levels rise above accepted limits.

Video Control. Visual monitoring is required to detect incidents in the tunnel that are not alarmed by the fire detection, emergency telephone or incident detection systems. An attentive operator watching the video monitors may locate an incident before the incident detection system can be activated. The operator could then initiate safety or corrective actions at the earliest possible time. In the event an incident is visually detected, the operator is able to manually control a certain camera's pan, tilt and zoom (PTZ) function to visually determine the severity of the incident before taking appropriate action.

Visual monitoring of the tunnel is accomplished by approximately 30 video cameras. All cameras are monitored simultaneously on five video monitors mounted above the operator's console. These monitors use split/composite screens to present video from multiple cameras on a single monitor. One monitor is devoted to each portal and one monitor is devoted to each directional cell (eastbound, center HOV and westbound). Video recording occurs on all screens displayed at the control console by five independent videocassette recorders. These units normally run in an elapsed time recording mode.

Approximately half of these cameras are fixed and the other half have PTZ capability (in a tunnel, roughly every other camera has PTZ function). PTZ control is possible by manual operation of the console and/or automatic preset positioning. In manual mode, the operator can select any of the PTZ cameras and control these functions through the tunnel computer keyboard. Remote control of the video system from the TSMC is processed through the tunnel computer.

Carbon Monoxide (CO) Monitoring. Each traffic cell has three to four carbon monoxide detectors monitored by the tunnel control system. Included with each monitor are two alarm contacts (warning and alarm) and one trouble alarm. Carbon monoxide values are available for display on the host computer's video monitor, environmental control panel and carbon monoxide equipment rack. If the carbon monoxide concentration of an area within a traffic cell should exceed 125 parts per million (ppm), the normal fan speed control is overridden and the supply and exhaust fans for the cell are increased to maximum speeds. These settings are maintained until the carbon monoxide level decreases below 90 ppm at which time the normal control resumes operation. This occurrence causes an alarm to be generated and displayed on various tunnel control systems along with a printout of the incident. A trouble alarm from a carbon monoxide monitor is displayed on the tunnel computer monitor. A faulty sensor causing false alarms can be inhibited by the operator.

Power Loss. In the event that a power failure occurs on both of the incoming utility lines, a standby generator system is activated. This system is sized to provide an adequate power supply to operate the ventilation fans at reduced speeds. Upon power loss, the VFDs will trip, thus cutting off all power to the fan motors. Once standby generators are on line, the PLC will generate a "reset" and "start" signal to the VFDs that restarts certain fans according to other tunnel variables.

The loss of both tunnel utility feeders would also result in extinguishing all lights in the tunnel except for the emergency lights. The emergency lighting panels are provided with uninterruptible power supplies (UPSs) to insure continuous operation in case of power failure. This power loss automatically initiates the start and synchronization of the generators. Once generator power is available through the UPSs, the night-time, day-time, and Step 1 (normal portal brightest on an overcast day) lights are activated. Maximum tunnel lighting on standby generator power is Step 1.

Tunnel Cell Fire. Fires within the tunnel are monitored by the fire control system that operates independently of the tunnel computer system. It provides fire alarm and location information to the tunnel computer and also generates a fire alarm input to the ventilation fan PLCs. The tunnel computer and operator may then decide on proper signal head and sign control or rely on the automatic settings as mentioned previously. Backup controls are available on the environmental control panel if the tunnel computer fails.

When the emergency is cleared, the operator will reset the fire alarm signal and the PLC logic will automatically return the fan speed control to normal.

Manual Operation. As mentioned above, each fan is capable of being operated under manual control, independent of other fans. This control may come from the tunnel control console operator, the environmental control panel or from the TSMC. Fans may also be controlled locally from their individual VFDs. Lockout for maintenance purposes is possible locally at the fans.

Traffic Control. Traffic signal heads, neon portal signs and variable message signs (VMSs) are used to alert traffic to maintenance work or lane obstructions in the tunnel. Critical incidents requiring tunnel cell closure are handled by automatic or manual control from the tunnel control console.

A neon portal sign is located at each tunnel roadway entrance to inform motorists of an individual tunnel roadway closure. The sign can be manually activated by operator input at the computer. When activated, the sign flashes TUNNEL CLOSED followed by DO NOT ENTER. The neon portal signs are automatically activated by the PLC upon a signal from the fire system.

Upon initiation of a signal head lighting sequence in response to a tunnel fire in a given cell, all signal heads in the direction of upstream traffic simultaneously turn to a steady amber. After five seconds of steady amber, the neon portal signal are activated to announce tunnel closure, simultaneously with all upstream signal heads turning red. All signal heads downstream of the incident area remain de-energized. The control operator is able to manually override the PLC signal head logic from the tunnel control console or the TSMC. After the emergency situation is resolved, the operator can manually turn signal heads to green and the signals automatically return to the normal "off" condition in 30 seconds.

During normal operation, the VMSs are monitored and controlled by an operator in the tunnel control room through the VMS central controller. Capabilities available through this mode of control include:

- Creating and storing messages on the VMS central controller hard disk;
- Retrieving stored messages from the disk for review or modification;
- Initiating display of messages on the VMS signs;
- Causing messages to flash and controlling the yellow flashing beacons on the VMS signs;
- Viewing the current status of each sign including what message is currently being displayed;
- Scheduling messages for display on a time-of-day, day-of-week basis; and,
- Requesting a failure report for all signs in the system.

Additionally, the operator may view the status of the signs through the tunnel computer console. Backup control of the VMSs is available through the tunnel computer console in the event of failure of the VMS central controller.

Traffic Data Stations. Traffic data stations located within the tunnel cells and at ramp entrances monitor traffic conditions by accumulating traffic counts, and computing average speeds and occupancy. These data are sent to the tunnel computer every 20 seconds. The tunnel operator/TSMC may then access this information through the console and through daily reports. Incident algorithms, executing periodically on the tunnel computer, alert the operator when the relative traffic measures reported by adjacent traffic data stations suggest the presence of an incident. These algorithms are not able to detect all incidents, but can give some indications during heavy traffic periods. Neither the traffic data stations nor the incident algorithms control traffic lights within the tunnel.

In addition, a system of high-frequency induction loops on the roadway are used to detect vehicles magnetically. The operator can use the video camera system to monitor these roadway loops so that the control room can detect anomalies in traffic flow for their investigation.

Light Monitoring. Threshold, transition and interior zone luminaires are controlled in such a manner that the light levels within the tunnel can be varied in intensity by steps to suit outside conditions (for example, bright sun, time of day, cloudiness, darkness, etc.). Control is effected by outside photo cells sensing ambient light levels. Photo cells are also included inside the tunnel to sense overall interior light levels. This arrangement allows lights to be switched "off" when lamps are new and the tunnel surfaces are clean. As the surfaces get dirty and the lamp output reduces with age, lights can be turned "on" to compensate for this loss and thus maintain interior design levels. This type of control is energy efficient lighting installation since it does not overlight the tunnel interior.

Conclusions

The Seattle tunnels were designed with safety in mind — based on the creed "Prevention, Detection and Control/Response."

Prevention. Roadway geometry was carefully designed to conform to safe highway standards. Tunnel lighting design emphasized

enhanced visibility for drivers as well as economy in construction maintenance and operating costs. A UPS system prevents a total blackout of tunnel lighting if power fails. Traffic lights inside the tunnel and VMSs outside were provided to stop traffic if an emergency occurs in the tunnel and to warn drivers approaching the tunnel of any problems. And, while hazardous cargoes could not be prohibited, systems to detect and control fires were installed.

Detection. The project's prime device for detection is a video monitoring system with PTZ capability and a control panel (designed for ease of use by the operator). Ceiling-mounted fire detectors tied directly into computer system automatically activate fire control systems. SOS boxes installed on each tunnel wall every 350 feet — complete with emergency telephone, fire pull and fire extinguisher — aid in incident detection. Two-way radio antennae were included for emergency vehicles to communicate with their headquarters, each other and the tunnel control room. Roadway loops spaced at 600 feet in lane are tied to an algorithm in the computer program designed to warn the operator of a stoppage or other anomaly in traffic flow.

Control/Response. A control room equipped with a computer provides an operator with the current status of all safety systems and with the ability to monitor and control all systems for emergency response. The tunnels' zoned sprinkler system automatically (with manual override) responds to fires. SOS boxes with fire extinguishers can be used by tunnel patrons. Carbon monoxide sensors warn the control room of excessive levels and automatically increase ventilation to provide more air.

The integrated system requires only one operator who oversees tunnel ventilation fans, the video monitoring system, fire pull boxes, heat detectors, the automatic/manual foam sprinkler system and the roadway loops to monitor traffic. While the tunnel safety system can function automatically to facilitate incident management, an alert and well-trained control room operator, capable of good judgment in critical emergencies, is still necessary. It is incumbent upon the tunnel manager to provide this type of personnel and to conduct exercises and training on a regular basis, where an emergency is simulated and all aspects of response personnel (police, fire, rescue) and equipment are involved.

Using state-of-the-art tunnel technology, the First Hill lid and the Mount Baker Ridge tunnel/lid have been equipped with the most modern and technologically advanced mechanical and electrical safety systems available today.

ACKNOWLEDGMENTS — *Sverdrup Corp. was project designer. FENCO Engineers of Toronto, Canada, designed the lighting system for both tunnels. A DEC Microvax serves as the host computer.*

PHILIP E. EGILSRUD *is a Sverdrup Corp. Fellow in Mechanical Engineering. He is a recognized expert in the design, development and management of electrical and mechanical systems for tunnels, specializing in ventilation, fire protection and electrical systems. With over 44 years of experience, he has served as Project Manager or Technical Advisor for all of Sverdrup's major tunnel projects. He served as Sverdrup's Project Manager for the I-90 Mount Baker Ridge and First Hill Lid tunnels, and as Project Manager for Sverdrup's mechanical and electric contracts for Boston's Central Artery/Tunnel project.*

GARY W. KILE *specializes in tunnel ventilation, fire protection systems, and utility piping design, providing construction inspection and software development. With over 20 years' experience in mechanical engineering, his background is in the management and design of highway tunnel projects. Joining Sverdrup in 1984, he has served as Deputy Project Manager for the I-90 Mount Baker Ridge and First Hill Lid tunnels. Currently, he serves as Sverdrup's Lead Mechanical Engineer for Boston's Central Artery/Tunnel mechanical and electrical contracts.*

The Charles River Basin

Converting a foul and hazardous estuary into an urban wonderland attests to the ingenuity, determination and thoroughness of two turn-of-the-century engineers.

H. HOBART HOLLY

There is no better example in the country of the value of the National Historic Civil Engineering Landmark (NHCEL) program than the Charles River Basin. Few among the millions who visit this famous site annually recognize it as an engineering achievement. Only those who read the NHCEL plaque on the wall of the Museum of Science overlooking the basin note its distinction.

The public views a beautiful water recreation and park facility, and rightly enjoys the outstanding landscaping. Visitors have little idea that this pleasant place once was a foul and unsightly tidal estuary — an intolerable situation that demanded correction. It was the engineering profession that solved those problems and made possible the Charles River Basin that the visitor sees and enjoys today.

Background

In early times, large expanses of salt marsh occupied both the Boston and Cambridge sides of the estuary. Through the years, these salt marshes were filled in and development took place on the new solid ground (see Figure 1). The Massachusetts Institute of Technology is located on some of this filled land. An area of some 675 acres of salt water with a mean tide range of 9.6 feet was left after filling in the Cambridge side and Back Bay in Boston in the nineteenth century. At low tide there were vast areas of odiferous and unsightly mud flats. To make matters worse, and positively unhealthy, drainage and even sanitary sewage discharges added to the pollution of the estuary. In addition, flooding at times of extra high tides posed a serious problem for people on the filled land.

Everyone agreed that the situation in the basin had to be rectified, but there was no agreement as to how it should be done. Many proposed ideas based on their own specific interests. Few viewed the problem as a whole. The major challenge at this time was the solution to the basin's engineering problems, and not the creation of a park that the engineering solutions would make possible.

Approaching the Problem

In 1891, Boston's Mayor Matthews spoke out for damming the Charles River in order to create the opportunity to have the finest water park of any city in the country. This park would be patterned after the Alster Basin in Hamburg, Germany. A group known as the Committee on the Charles River Dam was the principal advocate for the dam. Shoreline property owners formed the Beacon Street Committee to represent their interests. The Massachusetts Legislature had funded an investigation into the problem of the river from the Charles River Bridge

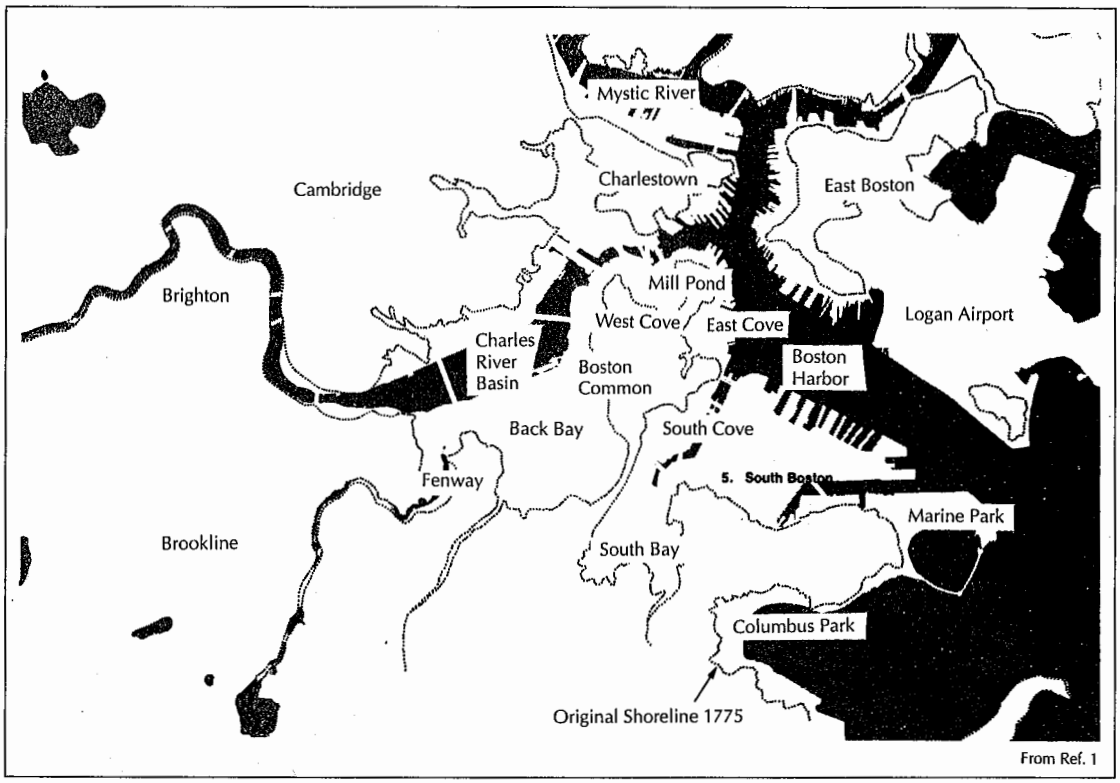

FIGURE 1. Map of the Boston metropolitan area showing the extent of land filling.

to the Waltham town line. The 1893-94 study favored a dam and a fresh water basin. This plan brought howls of protest from the Beacon Street Committee. The committee envisaged that the proposed fresh water lake would accumulate even more pollution and sewage since it would not be subject to the estuary's tidal cleansings. This possibility so alarmed the public that no action was taken on the matter. With the serious problem still in need of solution, another study was funded in 1901. The man selected as Chief Engineer of the Study Committee was John Ripley Freeman, a choice that made engineering history.

Developing a Solution

By 1901 Freeman had risen to the top of his profession as a consulting engineer in various fields, but especially in hydraulics. He was associated with the eminent hydraulics engineer, Hiram F. Mills, in his first position after graduating from the Massachusetts Institute of Technology in 1876. Ten years later, he became Engineer and Special Inspector for the Associated Factory Mutual Companies of Boston. He gained prominence for his study of fire streams. He also acted as consultant on water power and water supply projects, and served on the Metropolitan Water Board of Massachusetts.

In addition to hydraulics, his consulting work covered safety and building construction, including earthquake-resistant design. His work on water power and related projects extended to many sections of this country as well as Canada, Mexico, China, Italy and the Panama Canal. He was a leader in establishing hydraulic laboratories and hydraulic engineering research in this country. No one could have been better qualified for the complex Charles River Basin undertaking.

The first matter that had to be resolved was the severe problem of sewerage and drainage. Established practices were used and adapted to the basin's very complex conditions. The resulting design removed all sewerage and drainage discharges, actual and potential, from the basin area. The remainder of the study ap-

proached the problems of converting a salt water tidal basin into a constant-level fresh water lake. This conversion encompassed many unusual technical and environmental considerations. Not only did existing problems have to be solved, but also new problems that might be created had to be anticipated and avoided.

The thoroughness of the study was masterful. Among the subjects covered were:

- Borings and soundings of the area;
- Geological study of the estuary and Boston Harbor;
- Study of the effects of the Charles River flow in scouring the channels of Boston Harbor and the effects on the harbor of changed flows caused by a dam;
- Pollution of the Charles River above the basin;
- The possibility of malaria from mosquitos breeding in pools that would become fresh water instead of salt;
- The chemistry of the river;
- Biological problems;
- Erosion experience elsewhere;
- Effects on the commercial canals, wharves, etc., that would be within the basin;
- Upland water flow;
- Rainfall and flooding;
- Subsidence of the land and harbor;
- Effects on groundwater levels; and,
- Effects of the existing tidal flows into the estuary on the summer air temperatures in Boston (readings from all over the city for a full summer showed none).

The magnitude and thoroughness of the study is evidenced by its 600-page length, plus additional appendices on the detailed studies by special consultants that are over 350 pages long. Not only did the report resolve many of the problems to be faced or imagined by the former opponents, but it also became a model engineering study in itself that is still recognized for its outstanding quality.

Construction

The selection of the consulting engineer for the basin construction, Frederic Pike Stearns, was also a fortunate one. On the construction, he worked with the famous landscape architectural firm of Olmsted, Olmsted & Eliot. This combination of talents resulted in an engineering creation that is exceptional for its beauty, a characteristic for which Stearns' subsequent engineering creations were also noted.

Stearns was a self-educated engineer who rose to the top of his profession through diligent application to his work and studies. He exhibited a remarkable ability to work with others on projects that resulted in great engineering achievements. As a young engineer, he held responsible positions with the city of Boston in the planning and construction of the city's sewerage and water supply systems. In 1886, he became Chief Engineer of the Massachusetts Board of Health and gained a celebrated reputation for his thoroughness and good judgment. His studies of stream flows and water storage in which he established the principles for the economic development of watershed yields were especially notable.

After the Charles River Basin project he engaged in a number of planning and construction activities in greater Boston and other places. His projects were often cited for their environmental beauty and the blending of engineering structures into their surroundings.

Construction planning and design started in 1903. The first construction contract was awarded in 1907, and the work was completed in 1910. The dam was, of course, the major engineering feature of the project. The dam structure incorporated a roadway, a large lock for commercial vessels, sluice gates and overflow conduits. A dam with a width of approximately 100 feet was proposed, but it was decided to extend the 488-foot width required by the lock nearly across the river. This decision required nearly seven acres of fill, with 5.7 acres usable for park purposes.

To construct the lock, 3.5 acres had to be cofferdammed off. For all phases of the construction, the extensive piledriving was made difficult by swift currents. A shut-off dam was erected during the earth filling stage in order to reduce the washing away. Smaller locks were provided for small craft. The large lock was steam heated to prevent winter freeze-up. The

eight sluice gates were 7.5 feet wide by ten feet high.

Conclusion

The Charles River Basin was a success from the beginning. In recent times, a new dam has been built and the lock arrangement was altered to accommodate large numbers of small craft rather than commercial vessels. Other changes were made for minor improvements and to adapt to new conditions. However, it basically remains an engineering monument to John R. Freeman and Frederic P. Stearns.

H. HOBART HOLLY *is Chairman of the History and Heritage Committee, Boston Society of Civil Engineers Section/ASCE.*

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The Development & Implementation of a Traffic Forecasting Model for a Major Highway Project

The key to designing a highway project with regional impacts is the continuous interactive application of traffic modeling, transportation analysis and engineering principles.

TIM FAULKNER & LEONID VELICHANSKY

The Commonwealth of Massachusetts extends west approximately 140 miles from the Atlantic coast to its border with New York state. There is only one major highway traversing the Commonwealth in the east/west direction — Interstate 90 (the Massachusetts Turnpike). This highway connects eastern Massachusetts and the city of Boston with other major population centers within and outside of the state borders. However, it bypasses the city of Worcester, located in central Massachusetts. With Worcester being the second largest city in New England, there is an obvious need to link it directly to other population centers of Massachusetts. Figure 1 shows the regional highway system in central Massachusetts.

State Route 146, primarily a four-lane, divided north/south highway, is a major link between Worcester, the Blackstone Valley, and southern New England. Route 146 also connects Worcester with Providence, Rhode Island, and its harbor. Lack of a direct connection to the Massachusetts Turnpike forces users of Route 146 to travel a circuitous route to access the turnpike. In addition, the portion of Route 146 between the Massachusetts Turnpike and its northern terminus at I-290 in Worcester (Brosnihan Square) is a two-lane facility operating under severe congestion, which makes this "gateway" into Worcester extremely unattractive. The situation is further compounded by substandard geometry at the Route 146 interchange with State Route 20 (located in the

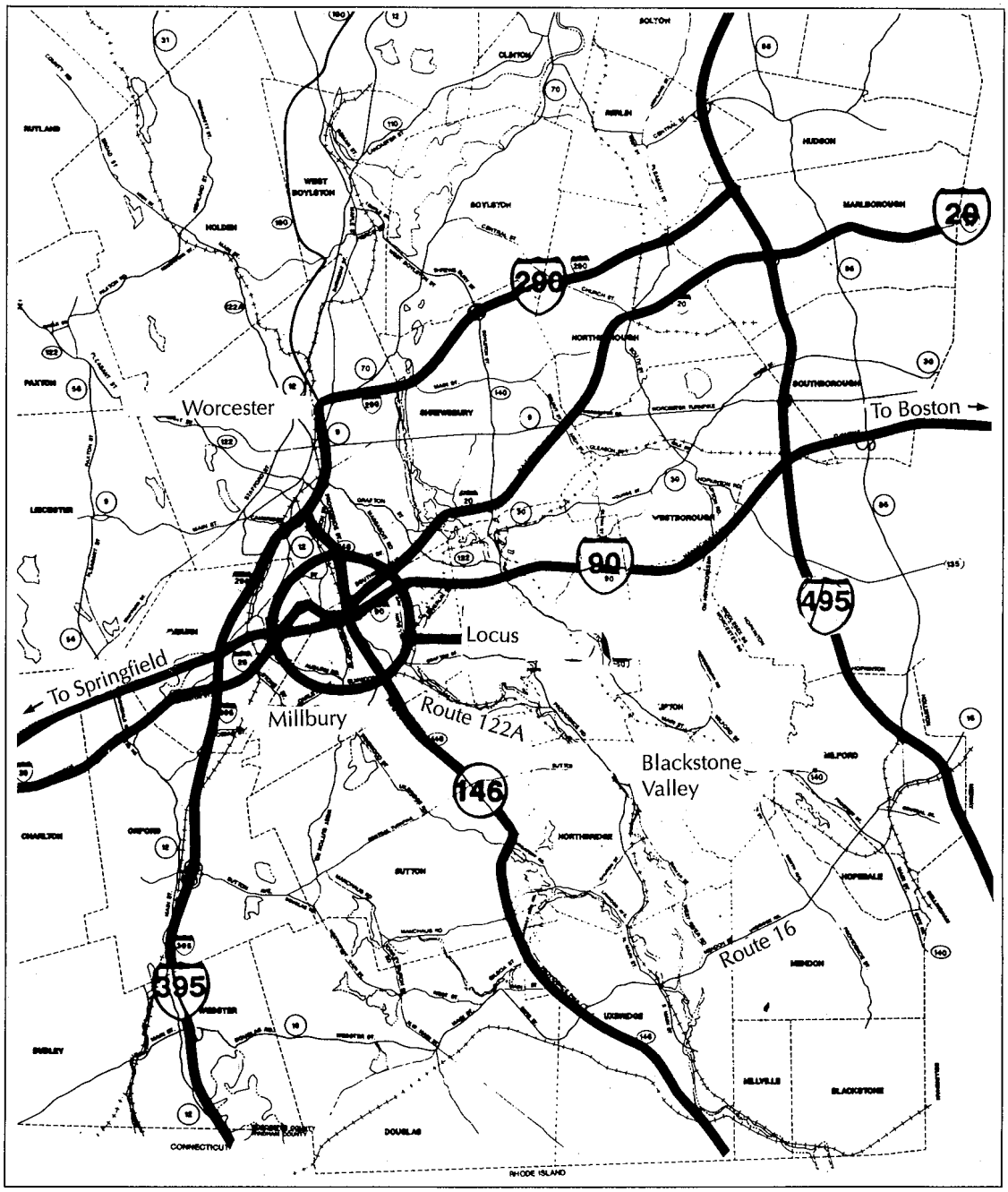

FIGURE 1. Location of the Route 146 improvement project.

immediate proximity of the Massachusetts Turnpike) and at Brosnihan Square.

Route 146 had been widened from two lanes to a four-lane limited access highway from the Rhode Island state line to the Massachusetts Turnpike in the mid-1980s. This widening has been complemented by similar work from the Rhode Island border to Providence and Interstate 95. The widening and improvement of Route 146 north of the Massachusetts Turnpike are considered necessary to address capacity and safety problems and to improve access to the Blackstone Valley and Eastern Massachusetts from Worcester.

Construction of a transportation link for Worcester to points east and west via a Massachusetts Turnpike interchange had been considered by the state since the 1960s. Worcester already has indirect access to the Massachusetts Turnpike through Interchange 10, but users going east to Boston must first travel west on Interstate 290 to reach that interchange. Due to engineering and financial constraints at the time, the idea for an additional interchange was dropped.

In early 1986 an engineering study was conducted to determine the feasibility of constructing an interchange between the Massachusetts Turnpike and Route 146. The study also evaluated the feasibility of upgrading a stretch of Route 146 from State Route 122A just south of the Massachusetts Turnpike northerly to Brosnihan Square. The study recommended widening of Route 146 between the Massachusetts Turnpike and Brosnihan Square from two to four lanes and reconstructing the Route 146/Route 20 interchange. The study did not recommend construction of a new Massachusetts Turnpike interchange. One of the conclusions of the study was that the new interchange would not generate enough traffic to justify its construction.

Due to the urging of business and community leaders in mid-1986, it was decided to investigate the feasibility of constructing a new Massachusetts Turnpike interchange and upgrading Route 146 in greater detail. The Route 146 Interchange with the Massachusetts Turnpike is intended to serve travel patterns to and from the Blackstone Valley, to Worcester to/from the east, and to Providence from points west along the Massachusetts Turnpike. An Environmental Impact Statement/Report (EIS/R) was authorized and an engineering consultant was retained by the Massachusetts Highway Department (MHWD) and the Massachusetts Turnpike Authority (MTA) to prepare the environmental document and the 25 percent design in 1987. One of the tasks of the EIS/R was to perform an in-depth analysis of the transportation system created through the implementation of the project, including impacts resulting from the potential economic development growth due to the project, also described as "secondary growth."

It was decided that the best way to evaluate transportation and economic impacts would be through a regional traffic forecasting model. Worcester's regional planning organization had previously developed and used such a model for other projects.

However, early discussions with the regional planning organization and a thorough evaluation of that model revealed that it was based on an estimated trip table. While the model provided sufficient information for regional planning purposes, it was decided that a greater level of accuracy would be necessary, given the sensitivity of the required analysis needed for the EIS/R. A desire to evaluate trip patterns, variations in these trip patterns with different roadway configurations and the required roadway geometrics dictated that an accurate trip table would have to be based on actual trip origins and destinations. Accordingly, the development of a new regional traffic forecasting model was initiated.

Model Description

In contemporary practice, traffic forecasting for large highway projects is generally performed by using a computerized traffic model. This procedure has been developed by the Federal Highway Authority (FHWA) and the Federal Transit Authority (FTA) over the past 25 years and has been copied throughout the world.

The travel demand model (TDM) selected for this project attempts to replicate "real-world" conditions through the use of mathematical models. Because there are more variables that influence travel behavior than can possibly be considered, most TDMs are basically approximations of real-world conditions, capable of reflecting major corridor level shifts in travel demand and magnitude. Recognition of the limitations of TDMs is a key factor in their appropriate application.

The TDM selected consists of a library of programs that provide the capability to perform the usual functions of traditional transportation planning with regard to trip generation, distribution and network assignment. Handling large regional traffic models with ease, the chosen TDM accommodates a network as large as 32,000 links (roadways), 8,200 nodes (intersections) and 2,000 zones. It also

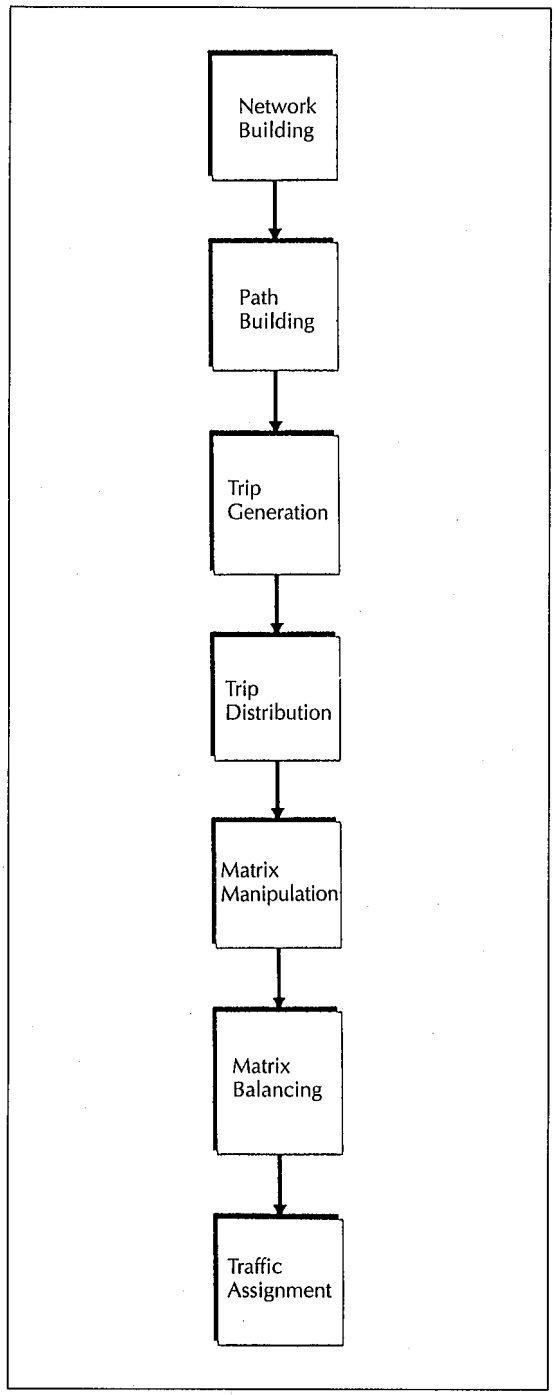

FIGURE 2. Normal system flow for the traffic modeling process.

has plotting capabilities that allows the user to output graphics files directly to a laser printer or plotter. The TDM software runs on a DOS-based personal computer, preferably 80386- or 80486-microprocessor-based personal computers with a math co-processor and 2 megabytes of memory and a graphics monitor (EGA or VGA). The time the TDM software takes to execute an iteration varies with the size of the network and the computing power of the computer. A project network that contains nearly 2,000 links, 900 nodes and 90 zones takes approximately five minutes to run on a 80386SX-based computer. An older 80386-based computer took approximately eight minutes to run the iteration, and an 80286-based machine took approximately 15 minutes.

The system flow for the software's modeling process is depicted in Figure 2. The TDM follows the traditional modeling process that begins with the user preparing link and zonal data files for input into the model so that a network can be built. Socio-economic data are then entered and trip tables are generated. Trips are then distributed across the roadway network. Next, the trip tables are converted into origin-destination tables and, finally, the trips are assigned to the roadway network.

Model Development

Before the model development began, a review of relevant previous studies, data and models was performed to determine what additional data would be needed to be collected. These data were evaluated in light of three time periods that were subsequently modeled: average daily traffic, morning peak hour and afternoon peak hour.

Study Area. The first step in the model development process was to determine the size of the study area. The study area had to be large enough so that the regional impacts of adding a new interchange to the Massachusetts Turnpike and its associated traffic diversions could be ascertained. It was decided that the study area would encompass approximately 600 square miles and include approximately 1,600 lane miles of roadway. The study area was bound by Interstate 495 on the east, Interstates 290 and 395 on the north and west, and Route 16 on the south (see Figure 1).

Data Collection & Organization. The next step in developing the TDM was to collect all the necessary data, including a roadway inventory, speed and delay measurements, traffic counts,

origin/destination survey, and the compilation of socio-economic data. The roadway inventory included interstates, urban arterials, local arteries and streets within the study area. These routes were traveled over a period of two weeks to determine the operating characteristics of each roadway (such as shoulder width, number and width of lanes, length of roadway segments, average running speeds, posted speeds and intersection configuration and control). This inventory was performed during all hours of the day to obtain representative conditions during peak as well as off-peak hours. This information was then used in calculating the capacities of the roadway segments and determining existing travel times throughout the study area.

As part of the effort to develop base year traffic flows, an intensive program of traffic counts was implemented to supplement historical data collected from various public agencies. Historical data included manual turning movement counts, vehicle classification counts and automatic traffic recorder counts on roadways throughout the study area. In addition, the MTA also provided daily and peak period volume information, including vehicle classifications, origin/destination reports, section density reports and traffic volumes at all interchanges over the past ten years. Additional manual turning movement counts and automatic traffic recorder counts were undertaken in the immediate project area to obtain the most up-to-date traffic volumes.

To establish the regional travel patterns within the study area and to develop statistics that were needed for the traffic model, an origin/destination survey was implemented. A roadside interview survey was conducted at four locations and a mail-in survey was conducted at two Massachusetts Turnpike interchanges. Motorists were asked for their origin and destination, purpose of the trip, trip frequency, what roadways they use or will use, and vehicle occupancy. When trucks were interviewed, their cargo and tonnage were also noted. For the mail-in survey on the Massachusetts Turnpike, toll personnel at two interchanges within the study area were instructed to distribute self-addressed stamped postcards to all traffic entering and exiting. Motorists were asked similar questions and then instructed to mail in their postcards. There was an approximate 12 percent return rate on the mail-in survey.

In order to organize the data from the approximately 2,300 motorists surveyed, the study area was subdivided into 54 traffic analysis zones (TAZs). These zones primarily followed United States census tracts, but where more detail was needed, United States Geological Survey (USGS) maps were used to determine concentrations of housing and industry within each town. Where census tracts had to be split, the percentage of the census tract in each zone was calculated by planimetering the appropriate areas. For zones in which there were a mix of industry and residential development, zones were loaded onto the roadway network in such a way as to account for driveway activity as well as access to residential neighborhoods. In addition to these TAZs, the 34 major entry and exit points to the study area were identified as external stations. Using the TAZ and external station framework, the survey data were processed to identify the TAZ or external station for each origin and destination.

Once the zonal structure was established, socio-economic data were collected for each zone. These data were collected by reviewing United States Census data, having discussions with the regional planning agency, interviewing town and city planners, and considering current land uses, historical development trends, community master plans and locally adopted zoning districts and ordinances. The socio-economic data that were collected included housing (stratified by income levels), population, and retail and non-retail employment. Also collected were projections on future land use within each zone.

Network Building. The modeling process started with the building of the roadway network. In this step, data for all the links and nodes were entered into the computer. Link data included beginning and ending nodes, link distance, link speed, link capacity and number of lanes. Nodal information included the X-Y coordinates of all nodes. The preliminary zone centroid loading points were also established at this stage. These points were

determined by reviewing the USGS maps and noting logical loading points. These data were then read by the computer, which produced a graphical representation of the input data.

Path Building. Once the network was "built," the next phase was to build minimum impedance paths from each origin zone to all the other zones. In this step, traffic had yet to be assigned to the network, and the only computing that was being done by the TDM software was establishing the shortest routes (based on travel time). Included in the path building was a turn movement penalty file. This file contained turning movement prohibitions (such as no left turns), penalty times for turning movements at stop signs and signalized intersections, and delays associated with toll plazas. For example, an impedance value of 45 seconds was assigned to left turns at a signalized intersection and a value of ten seconds was assigned for a right turn movement at an intersection controlled by a stop sign. Impedance values were assessed based on observed delay and traffic volumes for the type of roadway and area.

In addition, time penalties were added to movements through toll plazas to account for delays associated with toll plazas. To determine the time value of tolls paid, total transaction costs at each interchange were reviewed as well as the total traffic passing through the interchange. An average toll was arrived at, and then converted to delay by applying an hourly wage rate. This value was added to average delay through the toll plaza to calculate the total delay for the interchange.

Trip Generation. The next step in the modeling process was trip generation. During this step the actual number of zone-to-zone person trips made within the study area was estimated. Trip production and attraction is not synonymous with trip origin and destination. The basic unit of trip production is the household. For example, a trip from a household to a place of employment is produced by the household. The return trip to the household is still produced by the household because the means and the need for the trip is oriented to the household, not to the place of employment. Put another way, a place of employment does not really have the means to produce trips (*i.e.*, it has no automobiles or people), it can only attract trips that are produced by its potential employees.

By viewing travel from this perspective of productions and attractions rather than origins and destinations, it can be seen that the quantity of travel in any area is a function of its ability to produce trips and that trip production is controlled by the number and characteristics of the study area's households. Accordingly, the majority of the trips made within any area are termed *home-based* trips, with one end of the trip being the home. Under this definition both the trip to and from work are termed *home-based work* trips because the reason (production) for the trip is oriented towards the household. There are obviously trips that occur within any area where the home is neither the origin nor destination including, for example, trips from the work place to a store prior to returning to the home zone. Trips such as these are termed *non-home-based* trips. However, the "need" for this trip was created by the existence of the household, not by the existence of the store.

The results of the origin/destination survey showed that approximately 80 percent of the trips made during the survey period were home-based trips. This finding was not unexpected, since Worcester and its surrounding towns are highly urbanized and contain major shopping districts. In addition, many large corporations are located within the study area because of the preponderance of major roadways traversing the region.

The point is that the overall quantity of travel in any area is a function of its ability to produce trips, which in turn is a function of the number and characteristics of the households within each zone. The TDM algorithm used for the study provides trip production estimation procedures based on the number of households in the area and their average income distribution characteristics. Accordingly, the household and income data collected and organized on a TAZ basis was utilized to develop estimates of home-based work, home-based other and non-home-based trips based on the relationships set forth by the National Coorperative Highway Research Program (NCHRP 187) between income range and average daily person trips per household.[1] The trip productions estimated by the NCHRP 187 equations are based on income

levels in 1970 dollars, therefore these income levels had to be adjusted to 1988 levels through the use of the consumer price index to estimate inflation from 1970 to 1988. Once the trip production estimates were developed, the attractions for these produced trips were estimated. The TDM algorithms utilized employment levels (total, retail and non-retail) within the analysis zones in order to estimate zonal attractions. This information was collected at the early stages of the study.

After the preliminary estimates of trip productions and attractions were conducted, the next step was to add trips associated with *special generators.* Special generators are concentrations of activities of such size or unusual nature to warrant special consideration in trip generation analysis. Within the Worcester area there are three large universities (Holy Cross, Clark University and Worcester Polytechnic Institute) and a major medical center (the University of Massachusetts Medical Center). In addition to these institutions, there are other smaller colleges and universities. All educational and medical facilities within the study area were considered special generators. Within the study area there were 131 educational facilities, ranging from elementary schools to colleges and universities, and 53 medical facilities, including hospitals, nursing homes and clinics. The number of person trips that were produced and attracted by each of these special generators were then added to the zonal socio-economic data.

The final step in trip generation was to add in traffic counts from the external stations. Production values (traffic entering the study area) were accumulated as the external/internal productions, and the station attraction values (traffic that was exiting the study area) were accumulated as the internal/external attractions. These production and attractions were also added to the originally computed production and attractions.

After all production and attraction data had been input into the computer, the attractions for all internal/internal purposes were scaled so that the purpose attraction total matched the purpose production total. Non-home-based productions were then set equal to non-home based attractions on a zone-by-zone basis. External/internal station productions were balanced with external/internal attractions and internal/external station attractions were balanced with internal/external productions. The production and attraction balancing was necessary because it is possible that trip attractions could fall below or exceed trip production for a specific zone. The TDM methodology ensures that trip productions will equal trip attractions (since the ability to produce trips controls the overall trip-making characteristics of an area). The end result of this procedure was an estimate of the total number of trips produced and attracted within the study area.

Trip Distribution. Once the number of trips produced and attracted had been estimated, the distribution of these trips was determined. The TDM algorithm is based on the extensively used gravity model, which is based in concept on Newton's Law of Gravitation (gravitational force is directly proportional to the mass of any two objects and inversely proportional to the square of the distance separating these objects). This relationship has been applied to travel demand relationships for nearly 30 years and equates trip attraction or production potential as a surrogate for mass and travel time distance.

Over the years, it has been demonstrated that distance relationships are not consistent for all trip purposes, hence the standard procedure of categorizing trip production and attractions by purpose has evolved. In general, it has been found that travel time has a variable effect on trip interchanges that can be generally categorized as being less important for work trips than for other trips. These mass and distance relationships are reflected in the TDM procedure in the form of *friction factors* that represent the impedance associated with any zone-to-zone interchange. Impedances were based on travel time calculated using airline distance between TAZs in the study area. Using the zone-to-zone friction factors (impedances) derived from this procedure, the iterative process of calculating trip lengths and distributions was employed. Upon completion of this exercise, zone-to-zone trip interchanges were established.

Matrix Manipulation & Balancing. The next steps were the manipulation and balancing of

the trip table. The person trips that were calculated in trip generation and distribution steps were converted to vehicle trips by applying a vehicle occupancy rate for each type of trip and then all purpose types were added together to form one production/attraction trip table. The vehicle occupancy rates used in this study were derived from the previously described origin/destination survey.

In addition, the external/external trips were added to the trip table to account for trips crossing the study area. The number of through trips were determined by reviewing the results of the origin/destination survey as well as the Massachusetts Turnpike origin/destination data. The production/attraction trip table was then converted to an origin/destination vehicle trip table. The conversion could be performed in two ways — daily-to-daily or daily-to-peak hour. To convert to peak hour trip tables, a scaling factor was applied to the daily trip table. The scaling factor was arrived at by reviewing automatic traffic recorder counts and calculating the percentage of peak hour traffic as compared to the total traffic and then determining directional distribution on various roadway segments.

Traffic Assignment. With the final origin/destination matrix in place, the last and most complicated stage is traffic assignment. Three basic traffic assignment processes were considered:

- *Capacity restraint* with several iterations of all-or-nothing, which assigns all trips to the shortest path (if more than one shortest path exists, only one will be selected).
- *All-shortest-paths,* which assigns trips to all the shortest paths equally if more than one shortest path exists.
- *Stochastic,* which assigns trips to all efficient paths according to their probabilities of being used, based on the difference in impedance.

Traffic assignments were run using all three methods. The capacity restraint method with all-or-nothing iterations yielded the best results. This analysis was accomplished by comparing the volumes predicted by each type of assignment versus the actual volumes.

The capacity restraint process with several iterations of all-or-nothing is an iterative procedure in which the traffic assignment is run until all paths from one point to another are approximately equal. During the first iteration, all-or-nothing assignment loads traffic onto the highway network according to the shortest path (based on travel time) between two zones. Once all traffic is assigned, the link impedances are updated to reflect the fact that the shortest path may be over capacity and may not, in fact, be the shortest path any longer. New paths are then built according to the new link impedances.

This assignment process was run with six iterations that gave a fairly balanced network. The theory behind this process is formed on the following scenario. A user first selects a path along the route which is believed to be the minimum time. But other users also use parts of this path, and the travel time increases. The user then shifts to a different path (as do other users). Once that path gets congested, the user selects another path (as others may also). This process continues until the user cannot find a faster path, and the travel time on the final path is about the same as it would be on the congested original path.

Model Calibration

Once the traffic had been assigned to the network, the model had to be calibrated. The calibration process is one in which the existing counted traffic volumes are compared to the volumes that the model generates. Due to the sensitivity of issues surrounding the project, the client set one criterion at the early stages of the project that the traffic volumes assigned by the model for major roadways be within ten percent of actual traffic volumes.

There are numerous ways to refine the network so that the assigned volumes more closely match the existing volumes. The first step was to check land use assignments to individual traffic zones. It was noted that some zones were producing or attracting a significant amount of trips and other zones were not producing or attracting enough trips. The socio-economic data in these zones were reviewed and revised if it appeared that the employment or household data did not seem to be realistic.

Next, the friction factors were adjusted to get a better distribution of traffic over a range of travel paths. Also, zone centroid loading points were checked and revised as needed to spread the loading of zones onto more than one link, or change the shortest path between origin/destination pairs.

The highway network was reviewed for coding errors. During this process, link capacity, lengths and speeds were examined and adjusted upward or downward depending on the difference in volumes. Finally, turning movement penalties were revised to accurately reflect delays at a given intersection. The calibration process continued until the initially established goal of ten percent accuracy was achieved for most links and intersection turning movements.

Model Implementation

Upon completion of the calibration process, the model was ready to be applied to address the specific needs of the project. Its key applications included estimating traffic projections, looking at different what-if scenarios and generating the data required for various environmental analyses.

Traffic Forecasts & Secondary Growth Assessment. In order to assess immediate and long-term project impacts, two analysis years were targeted — 1994 and 2014. An extensive interview program was undertaken with local and regional planners, town managers and major developers by an economist associated with the project. Based on the results of these interviews, future employment and housing data were compiled for the years 1994 and 2014 in a format similar to that used during the model development.

Since calibration of the existing conditions involved revisions to the original socio-economic data, the future socio-economic data files were created by modifying the existing data to reflect changes in zonal employment and households. The resulting 1994 and 2014 daily and peak hour traffic volumes represented the no-build scenario (future traffic volumes without the project). The roadway network was then modified to reflect various infrastructure configuration alternatives, which were first viewed from a system-wide perspective. Traffic assignments were performed for the following alternatives at the onset of the project:

- Alternative 1 — No-build.
- Alternative 2— Reconstruction of the Route 146/Route 20 interchange.
- Alternative 3 — Alternative 2, plus reconstruction of Brosnihan Square, widening of Route 146 between the Massachusetts Turnpike and Brosnihan Square from two to four lanes, and widening of Route 20 in the vicinity of the interchange with Route 146 from two to four lanes.
- Alternative 4 — Alternative 3, plus construction of the Route 146 interchange with the Massachusetts Turnpike.

As a result of the analysis of regional transportation impacts of each of these alternatives, it was determined that Alternative 4 most fully satisfied the project objectives of strengthening the regional highway system, improving infrastructure capacity and providing a safe driving environment. This conclusion was reached through a combination of conventional capacity analysis and evaluation of changes in regional travel patterns. "Select link" assignments (where the volume from one link is assigned to the network so that traffic origins and destinations on that link can be determined) were used to determine the travel shifts that would occur with the implementation of various system-wide alternatives.

One of the objectives of the study was to assess the impacts of a potential economic development spurt that could be triggered by the construction of the project. Once the configuration of the key project components was established, the build and no-build assignments were compared. The output from the model provided information on zone-to-zone travel times. TAZs that were likely to experience ten percent or more travel time savings due to the project construction were further evaluated for potential increases in employment. Availability of vacant land, zoning and the magnitude of travel time savings were the parameters used in this analysis. It was assumed that there would be no additional growth in housing due

to construction of the Massachusetts Turnpike interchange. The resulting estimates of additional employment growth, performed by economists, were input into the model and the assignment process for the build alternative was repeated. Thus, the final build alternative assignments incorporated traffic volumes induced by the secondary growth.

Once the final assignments were made, link and intersection volumes were reviewed and traffic analysis began. As the analysis was proceeding, it was found that a few intersections and links had unacceptable levels of service. It was then decided to grade separate the intersections and add additional lanes to the links that were found to be over capacity. For the design of two interchanges, because they are located within a mile of each other, various what-if scenarios were tested with the model to determine the most optimal design of the two interchanges. Numerous configurations were studied, with the final configuration being half diamonds at each interchange with access to and from the south at the southerly interchange and access to and from the north at the northerly interchange.

Application for Environmental Analysis. Besides data for conventional traffic capacity and level of service analysis, the model-generated output provided data required for various components of the environmental analysis. The data generated by the model for the alternatives under consideration (daily and peak hour volumes, operating speeds and capacities) provided the necessary input for air quality and noise analyses. Statistics on regional vehicle-miles travelled and vehicle-hours travelled were used in assessing the overall project impacts on energy consumption.

Since the project significantly affected traffic on the Massachusetts Turnpike (a toll road facility), its impacts on the turnpike revenue had to be analyzed. Using "select link" assignments, a turnpike interchange-to-interchange trip table was established for both the no-build and build alternatives. Due to the construction of the new interchange with Route 146, a significant redistribution of trips was observed. Evaluation of the projected revenues based on the trip tables for two scenarios indicated that no net loss of revenue would occur upon construction of the project. In fact, it was projected that the construction of a new interchange would generate additional monies for the MTA.

Application for Preliminary Design. Since the model was initially created as a regional transportation model with the goal of assessing the regional impacts of the project, it required further refinement in order to provide accurate data for preliminary design.

The roadway network in the immediate vicinity of the project (Route 146 between the Massachusetts Turnpike and Brosnihan Square) was revisited and it was decided to include a number of additional local roadways to render more realistic assignments at a microscopic level. The model calibration was verified to assure that the addition of new roadways did not affect the overall model accuracy. Specific attention was given to intersection peak hour turning movements, since they are key in determining the final roadway configuration.

Once the refined model was calibrated to the desired degree of accuracy (typically ten percent), a detailed analysis of roadway facilities was performed. The results of these analyses were used to determine exact interchange geometries, intersection lane configurations, storage lengths, types of intersection control, traffic signal phasing and timing, etc. The traffic volumes used in the preliminary design were the same as those that were used in the environmental analysis.

Other Applications. In order to verify the viability of the construction of the project and to assure safe and efficient maintenance of vehicular and pedestrian traffic during construction, a detailed construction staging and traffic management plan was prepared. The project model was modified to reflect each major stage of construction, which included street closures, capacity and speed reductions, roadway detours, etc. The resulting traffic volumes were analyzed and the necessary adjustments to the staging plan were made so that the plan minimized the adverse construction impacts to vehicular and pedestrian traffic and minimized traffic diversions to alternate routes through residential neighborhoods

The information contained in, and generated by, the model was also used for several

other transportation projects in the region. The model data were integrated into the following projects:

- An on-going I-290 study conducted by the MHWD to address both safety and capacity concerns related to I-290, as well as other issues such as access to encourage development in the Worcester downtown area, traffic flow and safety on local streets.
- The city of Worcester gateway project — part of the city's master plan concepts for making the city more attractive, user-friendly and efficient.
- Other physical improvement projects to major roadways leading into Worcester to eliminate the complexity of driving around the city while providing motorists with more data on destinations and locations.

Conclusion

The TDM proved to be a vital component of this regional transportation project. Its many applications served a variety of purposes throughout the shaping of the project. A continuous interactive process between traffic modeling, transportation analysis and highway engineering was paramount in designing a project that will greatly enhance the regional transportation system and provide the infrastructure necessary to support future economic development within the area. It is also a valuable transportation planning tool that can assist other agencies and planning organizations in making decisions affecting the transportation and economic future of central Massachusetts.

The project is now nearing the completion of the final EIS/R, which is scheduled to be published in April 1994. The project's 25 percent preliminary design is scheduled for completion in June 1994.

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