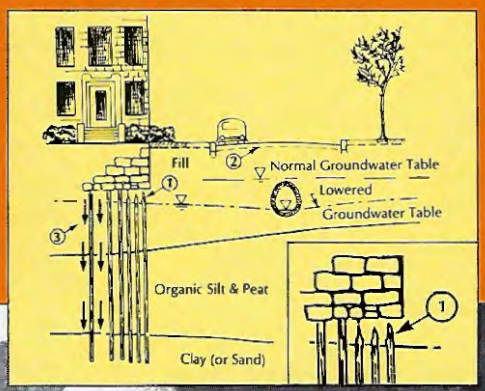
CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

FALL 1986 VOLUME 1, NUMBER 2 ISSN: 0886-9685

Back Bay Groundwater



Also in This Issue:

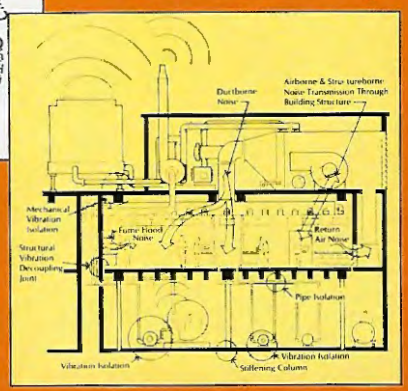
Lowell Waterpower System

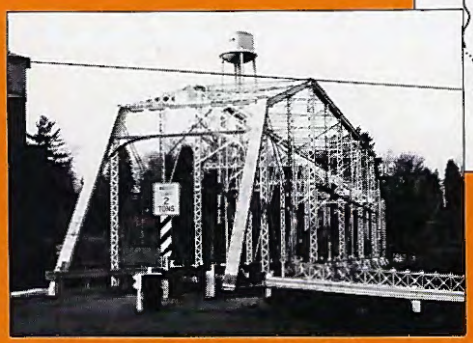
BSCES History

Vaiont Slide

Streamflow Distribution

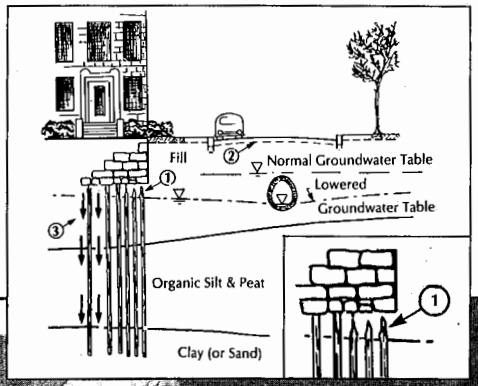
Semiconductor Clean Room Design





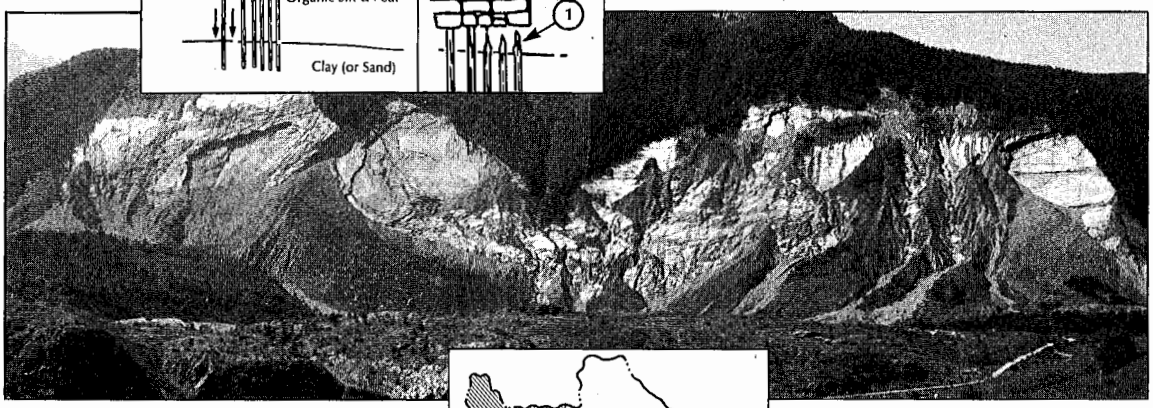
Bridge Rehabilitation

Back Bay Groundwater



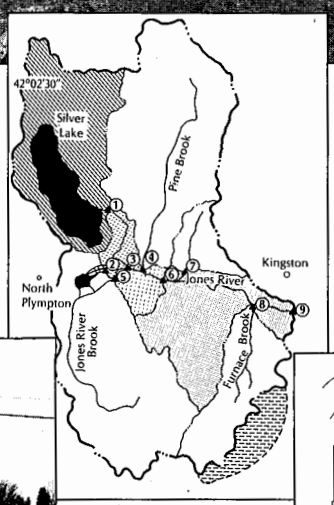
Also in This Issue:

Lowell Waterpower System
BSCES History

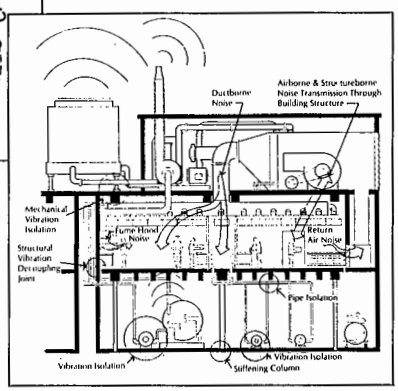


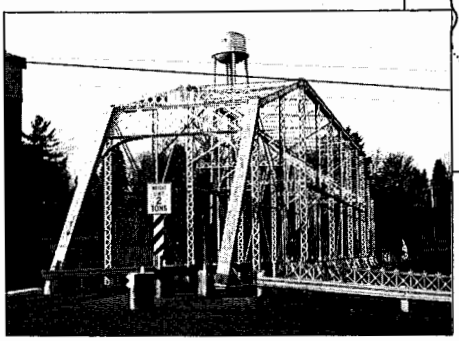
Vaiont Slide

Streamflow Distribution



Semiconductor Clean Room Design





Bridge Rehabilitation

CIVIL ENGINEERING PRACTICE • JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

CONTENTS

Building Technology for Microelectronics Clean Room Design — WILLIAM L. MAINI, MICHAEL K. POWERS & MARIO J. LOIACONO — 7

Clean rooms require especially tight environmental controls. Technological advances in air particle control and cooperative design and construction are key concerns.

Truss Bridge Rehabilitation Using Local Resources — ABBA G. LICHTENSTEIN — 27

Imaginative use of local resources including materials, expertise and labor can result in cost efficient projects.

Back Bay Boston, Part II: Groundwater Levels — HARL P. ALDRICH & JAMES R. LAMBRECHTS — 31

Man-made structures that permanently lower groundwater levels can have adverse effects on buildings with water table sensitive foundations.

A Geotechnical Analysis of the Behavior of the Vaiont Slide — A.J. HENDRON & F.D. PATTON — 65

In determining the safety of proposed reservoir slopes, engineers and geologists must have a thorough understanding of the causes of the Vaiont Slide.

Streamflow Distribution in the Jones River Basin — DAVID G. JOHNSON — 131

For streamflow distribution estimates to be more accurate, they must take into account hydrogeogical conditions as well as being based on surface-drainage area.

Lowell Waterpower System — H. HOBART HOLLY — 141

Not many projects have had such wide-ranging effects as this engineering project had on the industrialization of America.

BSCES: History & Heritage — GIAN S. LOMBARDO — 145

Few professional societies in this nation have had a continued history as the Boston Society of Civil Engineers Section/ASCE. Nor have many societies had so many members whose contributions to our nation's growth have been so far-reaching.

CIVIL ENGINEERING PRACTICE: JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE (ISSN: 0886-9685) is published twice yearly by the Boston Society of Civil Engineers Section/ASCE (founded 1848). Editorial, circulation and advertising activities are located at: Boston Society of Civil Engineers Section/ASCE, The Engineering Center, 236 Huntington Ave., Boston, MA 02115-4701; (617) 536-2576. 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 Boston, MA.

Subscription rates are: *U.S. & Canada* — Individual, $10.00/year; Library/Corporate, $15.00/year. *Foreign* — Individual, $15.00/year; Library/Corporate, $22.50/year.

Back issue rates are: *U.S. & Canada* — Individual, $6.00/issue; Library/Corporate, $8.00/issue. *Foreign* — Individual, $8.00/issue; Library/Corporate, $12.00/issue.

Back issues of *The Journal of the BSCE Section/ASCE* are available at $5.00 per copy, with the exception of Vol. 67, No. 4, which is $7.95 (U.S. & Canada), $8.95 (Foreign). Please make all payments in U.S. dollars.

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

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

Editorial, Circulation & Sales Office:

Civil Engineering Practice
Boston Society of Civil Engineers Section/ASCE
The Engineering Center
236 Huntington Ave. (617) 536-2576
Boston, MA 02115-4701

ERRATA

Regrettably, one of our Editorial Board members, Bruce Campbell, *TAMS*, was omitted from the listing of the Board members on page 5 of Volume 1, Number 1. Also, the caption for Figure 2 in "Community Participation in Public Works Projects," p. 26, should read, "The areas where different construction methods finally used for the Northwest Extension are shown. The tunnel entered bedrock about midway to Porter Square and exited just east of Davis Square. At Harvard and Porter Square stations the track is on two levels. The vertical scale is exaggerated ten times; actual inclines never exceed 4%." In "Immersed Tube Tunnels: Concept, Design and Construction," p. 68, col. 1, the first sentence in the third paragraph should read, "Preliminary estimates indicated that the concrete box design might be about $1 million less expensive than a steel shell design."

Building Technology for Microelectronics Clean Room Design

Clean rooms require especially tight environmental controls. Technological advances in air particle control and cooperative design and construction management are key concerns.

WILLIAM L. MAINI, MICHAEL K. POWERS & MARIO J. LOIACONO

FORTY YEARS AGO, a team of engineers at Bell Telephone Laboratories developed the world's first electronic digital computer, designated as the ENIAC (Electronic Integrator and Computer). This 30-ton, 20,000 square foot machine could perform up to 5,000 calculations per second, and required 50 technicians and 18,000 vacuum tubes to keep it operating.

Development of the integrated circuit as a replacement for the vacuum tube was the first step toward the miniaturization of computer components and the creation of the pocket calculator, microprocessor-controlled home electronics and personal computers. Today's microcomputers can fit on a desktop and routinely perform more than 1,000,000 calculations per second, an improvement by a factor of 200 times in less than four decades. Current research has developed experimental microchips from a silicon material capable of more than a million calculations per second — all in an area no larger than a baby's fingernail (see Figure 1).

Silicon integrated circuits are made up of elements called electronic switches or gates. These gates are individual line elements that can be as small as 0.1 micron (10^{-8} μ), with typical elements being one micron or about 1/100 the diameter of a human hair (see Figure 2). Working at this scale, a particle no larger than a human cell, residing on the photo negative used to print circuits, can create chips with blocked circuits. A particle this size on the printed chip can cause a short. Table 1 lists the gases, chemicals and metals commonly used in semiconductor manufacturing, as well as impurity sources during the production process and major production source compounds.

The facilities used to research, develop and manufacture these chips demand the strictest environmental controls and pose a complex challenge to today's technical building designer. Providing adequate air particulate and vibration/noise control, water treat-

TABLE 1

Production Materials & Impurity Sources

Gases Used During Production

Phosphine (PH_3)	1% in N_2
Hydrogen (H_2)	—
Nitrogen (N_2)	—
Helium (He)	—
Argon (Ar)	—
Oxygen (O_2)	—
Diborane (B_2H_6)	—
Arsine (AsH_3)	1% in N_2
Ammonia (NH_3)	99%
Silane (SiH_4)	15% in N_2
Hydrogen Chloride (HCl)	99%

Chemicals Used During Production

Sulphuric Acid (H_2SO_4)
Hydrofluoric Acid (HF)
Nitric Acid (HNO_3)
Hydrochloric Acid (HCl)
Acetic Acid (CH_3COOH)
Ammonium Fluoride
Phosphoric Acid (H_3PO_4)
Acetone
Methyl Alcohol
Xylene
N-butyl Acetate
1-1-1 Trichloroethane
Hydrogen Peroxide (H_2O_2)
Silver Nitrate
Potassium Hydroxide
Methylene Chloride
Dimethyl Sulfoxide
Chromic Trioxide
Cyclohexanol
Cyclohexalene

Metals Used During Production

Aluminum (Al)
Antimony (Sb)
Arsenic (As)
Barium (Ba)
Beryllium (Be)
Boron (B)
Boron (B_4C)
Carbide
Chromium (Cr)
Nichrome (NiCr)
Copper (Cu)
Gold (Au)
Indium (In)
Iridium (Ir)
Molybdenum (Mo)
Nickel (Ni)
Permalloy (Ni/Fe)
Supermalloy (Ni/Fe/Mo)
Pallaium (Pd)
Platinum (Pt)
Rhodium (Rh)
Silver (Ag)
Tantalum (Ta)
Tin (Sn)
Tin Oxide (SnO_2)
Titantium (Ti)
Titanium Oxide (TiO_2)
Tungsten (W)
Tungsten Oxide (WO_2)
Zinc (Zn)

Production Impurity Sources

Impurity Source	*Room Temp. State*
Phosphorus Pentoxide (P_2O_5)	Solid
Phosphorus Oxychloride ($POCl_3$)	Liquid
Phosphorus Tribromide (PBr_3)	Liquid
Phosphine (PH_3)	Gas
Phosphorus Spin-on; *i.e.*, Phosphosilica Film*	Liquid
Boron Tribromide (BBr_3)	Liquid
Boron Trichloride (BCl_3)	Gas
Diborane (B_2H_6)	Gas
Boron Spin-on; *i.e.*, Borosilica Film*	Liquid
Boron Nitride (BN)	Solid
Arsine (AsH_3)	Gas
Arsenic Trioxide (As_2O_3)	Solid
Arsenic Spin-on; *i.e.*, Arsenosilica Film*	Liquid

*Emulsitone

Source Compounds Used For Production

Silicon Tetrachloride ($SiCl_4$)
Trichlorosilane ($SiHCl_3$)
Dichlorosilane (SiH_2Cl_2)

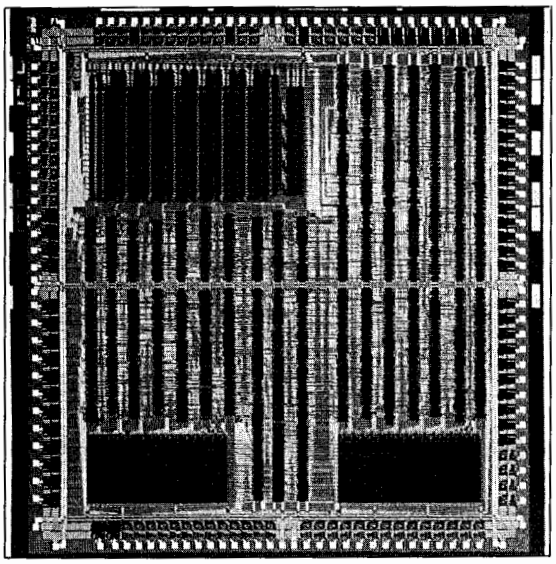


FIGURE 1. A typical integrated circuit.

ment for production rinsing of chips and waste removal systems are major considerations in clean room design.

Contamination Control

In the past decade, miniaturization has become a key concept in the semiconductor industry. Making electronic components as small as possible means faster operation of a greater number of circuits in a given space, resulting in the capacity to manufacture far more powerful and compact computers. At present, the technology for the control of particulate contamination in the research, development and production environment has raced beyond actual microelectronics needs. The industry is at a critical juncture, however, and this situation will soon reverse. Future circuit miniaturization will require further advances in contamination control technology.

Until recently, the state-of-the-art in integrated circuit line geometries was 2 micron line widths called Large Scale Integration (LSI) technology. Today, due to the market demand for one million transistors per chip, the development of Very Large Scale Integration (VLSI) technology is well under way. With line widths for these miniature integrated circuits typically as small as 0.1 micron, this new scale is also referred to as "submicron."

A mechanically controlled environment that regulates the quality of air is necessary for the production of these submicron components. In fact, the level of air purity is used to define the hierarchy of clean room design classifications. The standards for air cleanliness are outlined in Federal Standard 209B that specifies acceptable limits for airborne particle contamination, temperature and relative humidity ranges, differential pressure requirements and acceptable noise and vibration levels. Clean rooms are classified by their specific particle count definition. With five different classes — Class 10,000, Class 1000, Class 100, Class 10 and Class 1, the number of airborne particles 0.5 microns or larger in each cubic foot of air must not exceed the room class number. Particle count, particle size and sampling rate are the three classification parameters. Once the sampling rate has been fixed, the counts and the size constitute the clean room class (see Figure 3).

The issue of statistical error in classifying rooms as Class 10 or Class 1, the level required for future submicron work, has sparked a controversy over the validity of measuring 10

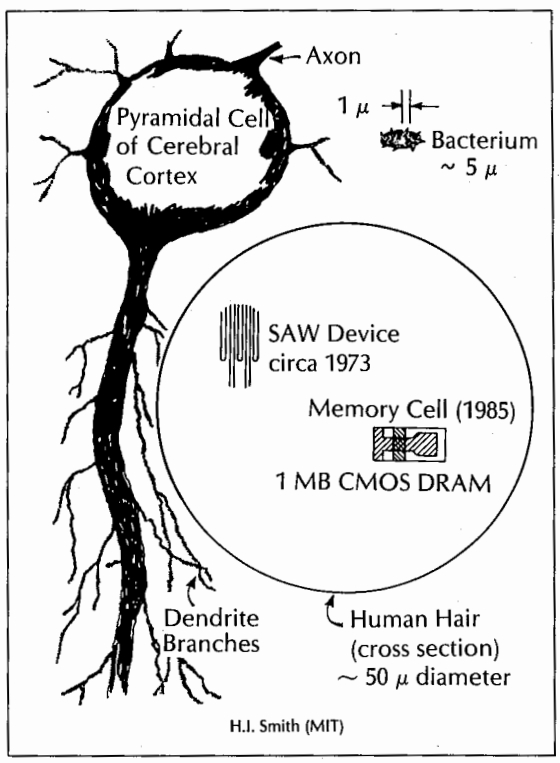


FIGURE 2. Relative size comparisons.

Definition of Terms

At Rest All production equipment in operation with no personnel present.

Chip (Microchip) A square of primarily silicon that is processed to ultimately be an integrated circuit.

Clean Room A clean room is an enclosed area employing control over the particulate matter in air with temperature, humidity, air flow patterns, air motion, pressure control and lighting as required. Clean rooms must not exceed the particulate count specified in the air cleanliness class.

Design Conditions The environmental conditions for which the clean space is designed.

Gate A circuit that has an electrical output dependent on its input.

High Efficiency Particulate Air Filter (HEPA) A filter with an efficiency in excess of 99.97% for 0.3 micron particles as determined by dioctyle phithalate (DOP) test.

High Efficiency Particulate Air Filter (VLSI) A filter with an efficiency in excess of 99.9995% for 0.12 micron particles.

Integrated Circuit (IC) An electronic device that performs the functions of many thousands of transistors.

Laminar Flow Air flow in which the entire body of air within a confined area moves within a uniform vertical air pattern along parallel flow lines.

Laminar Flow Room A clean room with a laminar flow requirement.

Large Scale Integration (LSI) Microelectronics manufacturing technology for integrated circuits with line geometries of 2-micron widths.

Line Width The width of a gate, or line, of a circuit in a microelectronic device.

Micron A unit of measurement equal to one-millionth of a meter (0.000039″). Twenty-five microns equal 0.001″.

Noise Criteria (NC) Curves These curves are a family of curves used to describe ambient noise levels in buildings using frequencies at dB levels. These curves have been developed from experimental data where each spectrum level represents approximate perceived equal loudness. The human ear is most sensitive at the 1,000-2,000 Hz level and is less sensitive at lower frequency levels.

Non-Laminar Flow Room A clean room that does not require a laminar flow pattern.

Normal Operating Conditions All production equipment, exhaust fans and air conditioning systems in operation with personnel present.

Particle Size The maximum linear dimension of the diameter of a particle usually measured in microns.

Unoccupied A bare room before production equipment has been installed with the clean room air handling equipment in operation, following the initial clean down period with no personnel present.

Very Large Scale Integration (VLSI) Microelectronics manufacturing technology for integrated circuits with line geometries as small as 0.1-micron widths.

Wafer A thin, flat circular disk of primarily silicon that is masked, oxide-coated, doped and processed for separation into numerous electronic devices or for packaging as an integrated circuit.

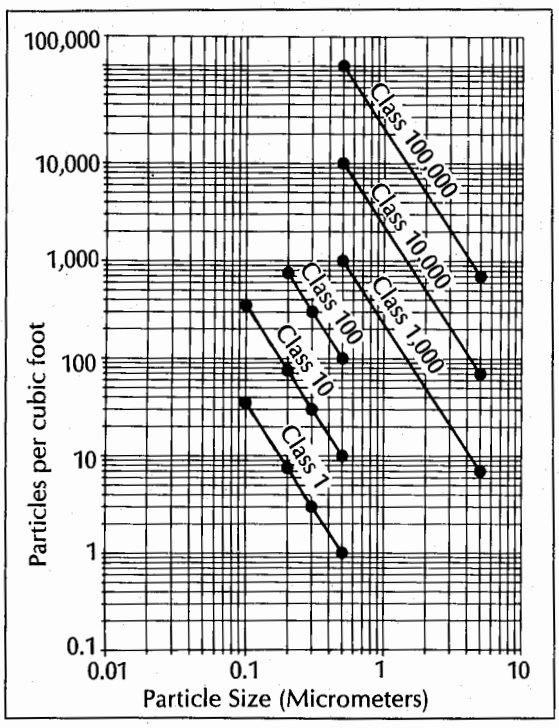


FIGURE 3. Proposed class limits in particles/cubic ft. (from revision to Federal Standard 209B).

or 1 particles in a cubic foot of air. Although these minimum particle conditions can almost certainly be created, they cannot be effectively measured. The current challenge to the contamination control industry is the development of technology and methodology that will produce accurate testing data for Class 10 and Class 1 clean rooms.

Environmental Characterization

Five major environmental characteristics determine the criteria for the clean spaces that will accommodate integrated circuit development. These are air flow velocity, particle size and number, temperature, relative humidity and differential pressure. Federal Standard 209B states that air flow velocity through a horizontal cross section of a laminar flow space must be maintained at 90 (±18) feet per minute throughout an undisturbed room. These measurements are typically taken at the ceiling just below the High Efficiency Particulate Air (HEPA) filter. Measurements may also be taken at table height (42″) and at the level of any air return grilles or perforated floor panels where the air leaves the clean room. These velocities will vary depending on the type of air return system utilized, *i.e.*, sidewall return or raised floor.

The actual number of airborne particles of certain sizes in a 1 cubic foot volume will characterize the class of the clean room space. There should be a definite correlation between air velocity and airborne particle counts. Typically, field testing has shown that air flow velocities as low as 75 feet per minute will effectively remove particles in the room. At air flow velocities of above 110 feet per minute, testing has shown that any improvement in particle control appears insignificant. The choice of the final operating velocity should represent a balance between the energy consumption for the overall facility and standards for room cleanliness. The typical electrical costs for fan energy for an average clean room that changes its air 600 times per hour (filtering it through a number of fine glass mats) are approximately $20/square foot each year.

Temperature ranges are established as demanded by the product and the personnel occupying the area. In a VLSI clean room, the desired temperature tolerance may be 68° (± 1°) F. Specific areas may be controlled to even closer tolerances of 0.18° F in order to minimize expansion/contraction effects on the manufacturing process.

Typical relative humidity requirements in a VLSI clean room are 37.5 (±2.5) percent. If the relative humidity in some clean room areas is higher or lower than the recommended values, a decrease in the success rate or yield of the final product may result.

A differential pressure requirement of 0.05 psi is recommended with respect to any surrounding atmosphere of the clean room area in order to impede the flow of contaminated air into the room.

These environmental characterizations of VLSI clean rooms are used to qualify the facility for manufacturing or research/development, and to assist the end user in implementing the necessary environmental monitoring programs. Proper clean room characterization enables the user to produce high density devices and to successfully predict

TABLE 2

RP-006-84T Test Areas

Uniformity
Filter Leak
Parallelism
Recovery
Particle Count
Particle Fall-Out
Induction
Pressurization
Air Supply Capacity
Lighting Level
Noise Level
Temperature
Humidity
Vibration

product yield.

Clean Room Design & Testing Standards

During the past two decades, the criteria for the certification of clean rooms have followed the recommendation of Federal Standard 209 and its subsequent revisions, designated as 209A and 209B. Specific details for performance testing have never been included in Federal Standard 209 or its updated versions — resulting in widely divergent test procedures. In 1970, the American Association for Contamination Control, now merged with the Institute of Environmental Sciences (IES), issued a Tentative Standard CS-6T that contained greatly expanded methods for the testing of clean room particle counts as well as for other environmental factors. CS-6T has not been distributed as widely as Federal Standard 209, and thus had not the same impact on the clean room industry.

In the meantime, Federal Standard 209B is soon to be republished as Federal Standard 209C in order to respond to the certification needs of Class 10 and Class 1 rooms. Tentative standard CS-6T will also be revised as RP-006-84T, introducing the methodology for 14 tests to review laminar flow and non-laminar flow clean rooms. Table 2 lists these 14 test areas.

Feasibility Studies

Successful microelectronics facilities are the result of well defined programming and intensive facility planning efforts that begin during the early design phases. In many cases, a feasibility study prior to the initial design is appropriate to determine proper site selection, overall facility size and function (see Figures 4 and 5). Since many facilities require extensive capital expenditures on the part of the owner, the initial step of preparing a well defined feasibility-programming document allows for a more accurate assessment of facility needs. Established site conditions, and design/construction schedules will then generate more accurate preliminary cost models.

Facility Considerations

Site selection is a critical factor and must be studied in detail to assess the stability of the external environment and the reliability and quantity of available resources. The most critical initial decision is the selection of a site with well documented vibration characteristics. Although many internal isolation systems can mitigate vibration, a greater degree of control can be exercised over an isolated site where vehicular traffic, railroad routes, high pedestrian activity and vibration-transmitting soil types are at a minimum. A relatively isolated site will also supply open air spaces into which emissions from scrubbed exhausts may be rapidly dispersed and diluted. This factor is particularly important for production facilities using high concentrations of arsine and phosphine. A concentrated use of solvents in the production process could lead to ambient odors unfamiliar to the surrounding community. Odor absorption equipment is available, but costly.

The availability and reliability of water, electrical power and clean outside air are significant factors in site selection. On-site water is used to feed the sophisticated deionized high purity water systems necessary to rinse process components. The initial purity of this local water source can affect the ultimate cost of these sophisticated water polishing systems. A reliable source of local

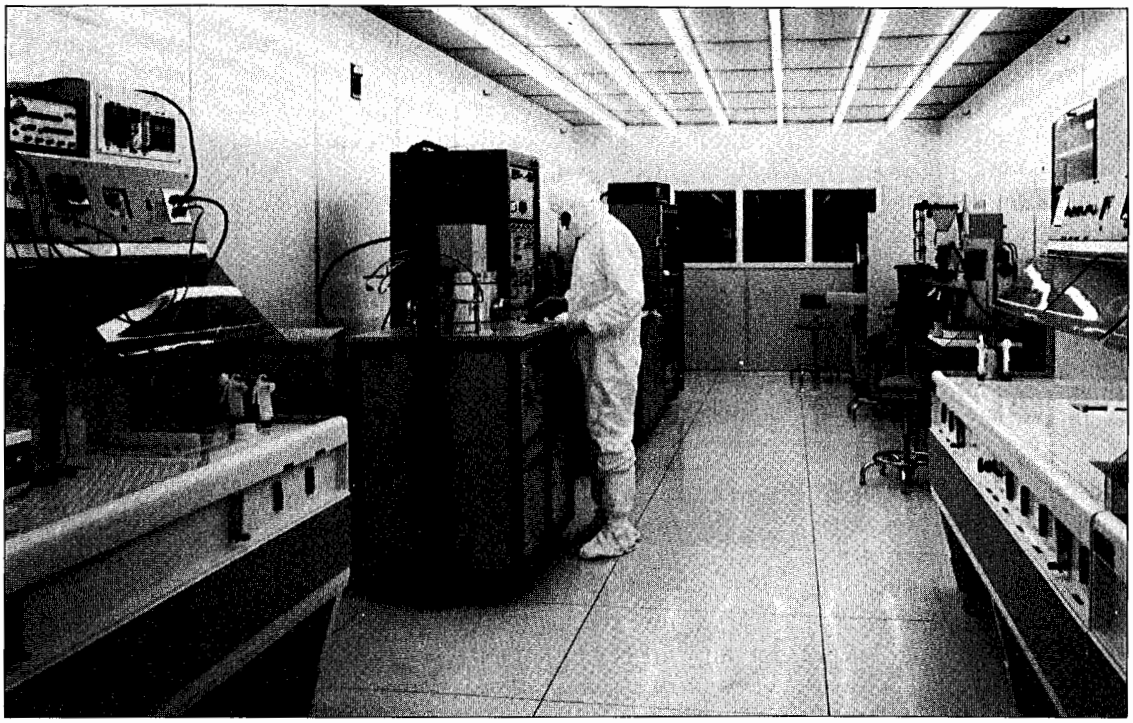


FIGURE 4. A typical clean room.

electrical power is necessary to sustain critical semiconductor operations without equipment or product loss. Back-up emergency generators are an expensive safety measure and areas subject to frequent power outages should be considered questionable sites. Large quantities of outdoor air will provide ample replenishment for the extensive exhaust systems supporting wet stations, fume hoods and equipment. Since wafer fabrication areas require the rigid control of humidity, temperature and outside air contamination, the air conditioning equipment must be designed to handle the "worst case" environmental conditions of outside air. As a result, sites in regions with wide climatic variations will require more expensive heating, ventilation and air conditioning (HVAC) support systems. During the feasibility study phase, the size, location and cost implications of the air-handling systems must be reviewed.

Mechanical, electrical and other manufac-

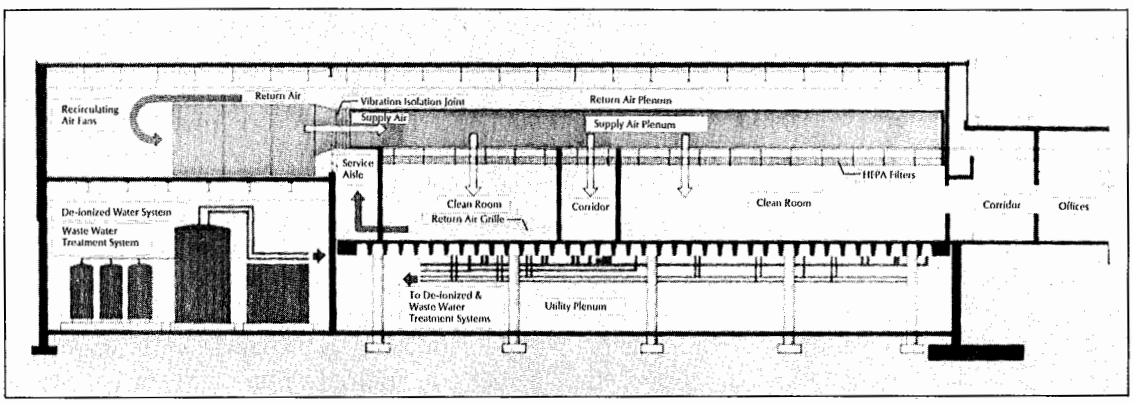


FIGURE 5. A building section from a typical microelectronics facility.

turing process portions of the project should be conceptualized at these initial stages. These systems costs can ultimately represent a range of 50 to 60 percent of the total base building construction cost and will account for a significant amount of the space allocations. Systems supporting temperature stability, relative humidity, air purity and air changes (up to 10 to 12 space air changes per minute in Class 10 and Class 100 rooms) as well as equipment-related process cooling water, process gases and electrical power must be identified early to determine the cost impact on the overall project.

Large semiconductor plants use substantial quantities of gases in the manufacturing process. Bulk storage of liquid oxygen, hydrogen and nitrogen, as well as bottled chemicals and specialty process gases, must be evaluated to determine impact on the building site and the surrounding environment. Even though a thorough review of these considerations may favor a specific location, it is important to review the environmental impact of the project and local community sentiment. Changing attitudes toward the environment may prevent future expansion of the facility. A thorough knowledge of the applicable laws and regulations will assist in moving the project ahead.

The well-developed feasibility study can generate a set of guidelines that will determine a well-structured project cost model. By addressing location, conceptual layout, image, clean room classification and area, construction schedule and available budget, a reasonable cost projection can be made. Cost models should include as many individual line items as possible and be flexible enough to allow for constant readjustment as the scope of the project develops more clearly. Microelectronics facility spaces can vary in cost from $125/sq. ft. for Class 100,000 to $750/sq. ft. for Class 10.

Regulations

Codes, permits and certifications covering microelectronic facilities have yet to be specifically formalized. The Semiconductor Industry Association has published their proposed revisions to the Uniform Building Code, which have been designated as H-6. These revisions attempt to formulate the requirements specific to the construction of microelectronics buildings, including smoke exhaust systems, process gas distribution and storage areas, ventilation of chemical storage areas and unique alarm requirements.

The insurance industry, particularly Industrial Risk Insurers and Factory Mutual, has also taken a leading role in defining safe construction standards. Basic design requirements have been continuously revised by this industry to provide better protection in the case of an emergency. For instance, fiber reinforced plastic ductwork is now internally sprinklered to protect the duct system from fires triggered by the use of silane gas and the presence of condensed materials. Instead of common return air plenums, recirculating air is now compartmentalized with segregated supply and return air, thus minimizing the spread of smoke and fire into other areas through the mechanical system.

Design Guidelines: Flexibility & Cleanliness

Vibration and noise control, particle control and facility flexibility are the major determinants of the design program for microelectronics facilities. Each determinant implies a net of conditions that influence architectural and engineering decisions.

The use of modular, factory-assembled room systems utilizing carefully manufactured high-quality components of non-shedding materials is making an impact on the industry. Still, the field-assembled room consisting of several prefabricated components integrated into a design persists as the most common approach. Many advanced projects of this type are constructed by specialty contractors, experts in this type of construction. The basic components of clean room construction can be summarized as follows:

- A laminar flow filter system at the ceiling with integral lighting
- Unistrut-type wall framing
- Prefabricated wall panels with smooth, baked enamel, porcelain or plastic laminate finishes
- Welded chemical-resistant conductive

flooring

- A perforated raised floor system (if required) appropriate for clean room use, or sidewall return grilles
- Coved flooring bases for ease of cleaning
- Appropriate glazing for outside awareness, visitor observation or safety
- Sidewall grilles with dampers, or dampers in the raised floor for room balancing
- Epoxy-coated walls in service aisle areas for ease of maintenance
- The use of appropriate entry configurations (air lock or component air shower)

The clean room design solution must respond to the need for non-disruptive change (flexibility), easy maintenance or cleaning on a periodic basis, non-shedding construction materials, and ease in coring services for equipment through walls and contamination control.

Vibration & Noise

A completely vibration-free environment is an unrealistic design objective for any sophisticated advanced technology facility. Generally, facility vibration criteria are projected from research compiled by manufacturers and specialty vibration consultants with regard to the manufacturing process' anticipated sensitivity to vibration. These criteria consist of various limits of vertical velocity amplitude *vs.* frequency (see Figure 6).

In this figure, the vibration-sensitive manufacturing equipment corresponds to lines A, B, C and D that are defined as follows:

A. Optical balances, bench microscopes

B. Aligners, steppers, etc., for 5 micron or larger line widths

C. Aligners, steppers, etc., for 1 micron or larger line widths

D. E-Beam and other 1 micron or submicron equipment; scanning electron microscopes

The common criterion for the submicron microelectronics facilities of today is the Sensitive Equipment D line. It is interesting to note that the working magnification level for

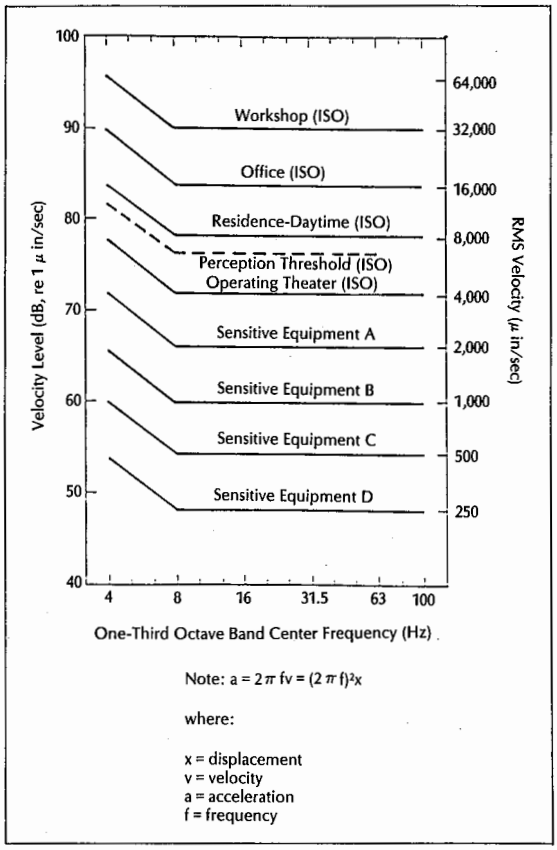


FIGURE 6. Vibration criteria for sensitive equipment in buildings (courtesy of BBN Laboratories, Cambridge, MA).

this equipment is approximately 28 times lower than human perception threshold limits.

Levels of vibration are controlled by utilizing structural designs with stiffness and mass. The most common system used today is the concrete waffle slab. The use of this system maximizes the stiffness requirement with its ribbed construction while also using more concrete in volume than structural slabs, thus creating significant background mass. The inside dome areas of 4- to 6-in. thick concrete allow enough flexibility for the penetration of the various support piping and ducts related to the clean room. The use of a waffle slab also offers the end user the added benefit of a 350 to 400 lb./sq. ft. load, the structural requirement for such heavy process equipment as E-beam and ion implantation equipment.

Pedestrian traffic (footfall) within the

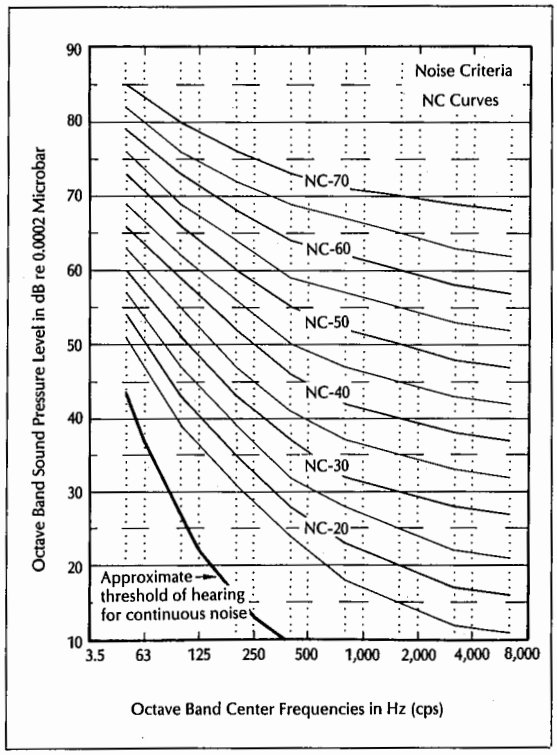


FIGURE 7. The criteria for mechanical system noise (courtesy of BBN Laboratories, Cambridge, MA).

facility is one of the most important vibration sources to consider. The most reliable solutions (other than administrative controls) are the structural separation of high traffic areas, or the use of heavier structural components to bring footfall-induced vibrations within acceptable limits.

Mechanical equipment vibration is usually mitigated by the use of spring isolators, inertia blocks, flexible connections on ductwork or conduit, and equipment selected for its rotating rather than its reciprocating components.

Noise is always a difficult problem in mechanically-intensive facilities, and microelectronic facilities are no exception. Since some noise is vibration related, the vibration features of a building assist in mitigating sound transmissions. However, air-handling systems, fume hoods, exhaust systems and equipment areas (chillers, boilers, pumps, etc.) must be carefully located and appropriate silencers or architecturally-related sound-absorbing elements (walls or casings) must be used to attain a relative sound level indicative of the potential uses of the spaces.

Figure 7 shows the Noise Criteria (NC) Curves that indicate sound levels in dB *vs.* frequencies. The NC-55 noise criteria curve is the generally acceptable clean room area design limit for noise within these spaces. Figure 8 identifies several problem areas that are associated with vibration and noise. A well planned facility will have design components that mitigate the typical issues shown.

High Purity Water

A critical element in the integrated circuit manufacturing process is the ultrapure rinse water used to wash the product at various stages of development. Water for microelectronics use may be purified by distillation, ion exchange, reverse osmosis, ultrafiltration, electrodialysis or a combination of these methods (see Figure 9).

A typical water purification or de-ionized (DI) water system consists of the following:

- Pretreatment/Make-up
- Final Purification
- Storage/Distribution
- Polishing Station

The pretreatment portion of the purification system treats the raw make-up water so that it is suitable to enter the final purification system. Included in this process is the filtration of suspended solids, removal of organics, pH adjustment to minimize scaling, UV sterilization and chemical treatment to prevent the fouling of the reverse osmosis membranes, pre-heating to 70° and reverse-osmosis. Pretreatment removes the majority of the dissolved salts, organics and bacteria. At this point, the treated water should have a resistivity of 1 to 10 megohms and be free of organics and bacteria.

Storage/distribution consists of one or a series of storage reservoirs used to handle peak flow conditions. The stored water is constantly pumped through the polishing loop and returned to the distribution pumps. Final purification consists of ion-exchange and resin traps, prior to final filtration in the polishing

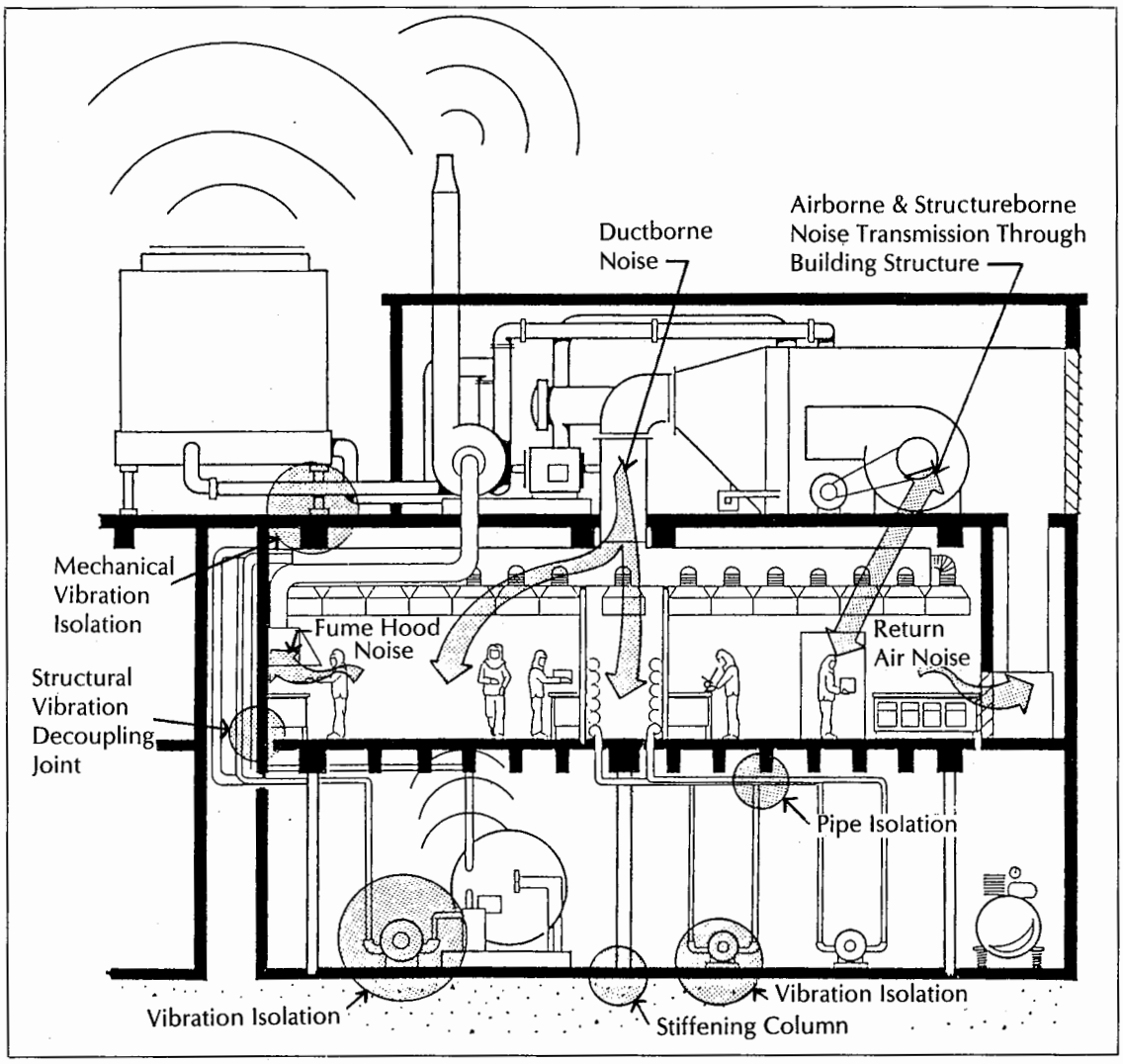


FIGURE 8. Vibration/noise problem areas within a microelectronics facility.

loop.

The polishing loop is composed of UV sterilizers that destroy over 99 percent of the remaining bacteria, polishing de-ionizers that raise the resistivity to 18 megohms, and final filters (for the sub-micron range). The filters prevent resin debris and dead bacteria from contaminating the distribution loop. The water is now considered "Electronic Grade E-1" with a resistivity of 18.3 megohms and can be used for process rinsing.

The state-of-the-art in piping material used to distribute DI water is polyvinylidene fluoride (PVDF). This material has replaced PVC, which was formerly popular due to its low cost and durability, after it was discovered that additives in PVC can migrate into the ultra-pure water.

Waste Water Treatment

Most waste water from the integrated circuit manufacturing process requires only neutralization prior to being discharged to local sewage systems. This process equalizes the strength of strong acids and bases in an equalization tank that overflows by gravity into a treatment tank. Caustic (sodium hydroxide) and acid (sulfuric) solutions are pumped into the treatment tank under the control of a pH probe via metering pumps.

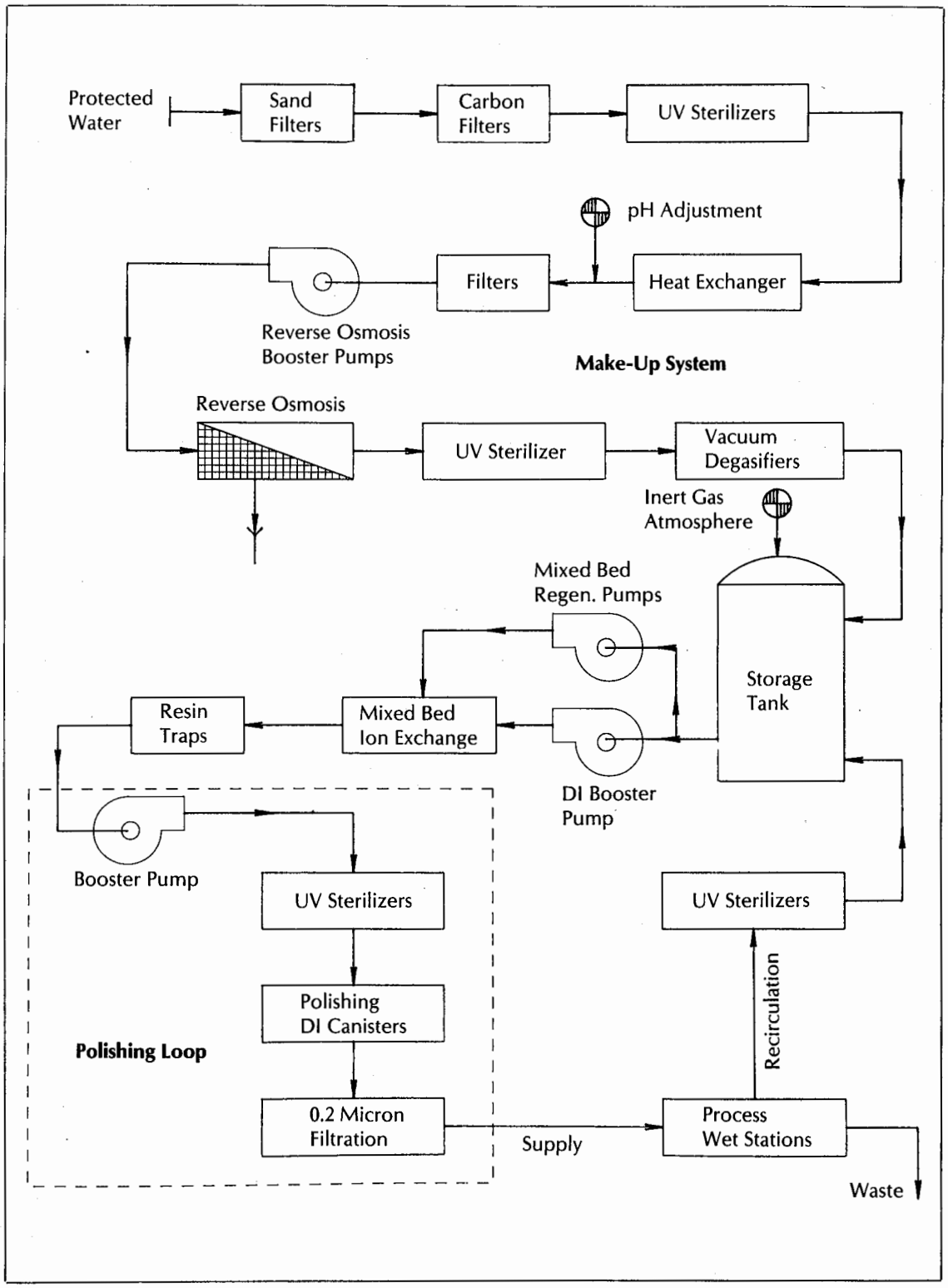


FIGURE 9. A schematic of a typical microelectronics facility high purity water treatment system.

This process is automatic and continuous (see Figure 10).

The overflow from the treatment tank flows by gravity to a wet well and is then pumped to the building sewer. The effluent pH is continuously monitored and recorded along with effluent flow. If the pH deviates from the set pH range (6.5 to 9.0), an alarm alerts the facility monitoring system.

Other liquid chemical wastes such as hydrofluoric acid (HF) effluent and solvents are normally collected in dedicated waste piping systems to on-site storage vessels for off-site disposal.

Contractor Prequalification

Microelectronics projects are among the world's most complex building types. Clean room construction demands exacting room installation tolerances, precise HVAC air-balancing, delicate HEPA filter placement, unusually low internal supply ductwork and process piping cleanliness specifications, room pressurization testing, clean construction sites and the total cooperation of owner, architect, engineer and contractor.

Prequalification of subcontractors by the building design team must be based on past experience and performance with microelectronics projects. This type of bidding approach requires that the owner and design team research and evaluate past subcontractural performance. Despite the great amount of effort this involves, such planning pays higher dividends in decreased risk and greater efficiency. A typical prequalification procedure follows:

- Identify potential qualifiers
- Assess past performance
- Evaluate the ability of potential qualifiers to complete the particular project scope

The prequalification of mechanical/process and clean room subcontractors should be a minimum requirement. Other subcontractors who could also be prequalified include electrical, fire protection and roofing.

Construction Procedures

A finely-tuned construction control procedure is integral to a successful working facility. Clean rooms must be totally sealed throughout construction to prevent the entrainment of large particles into the protected space through areas other than the HEPA filter supply systems. The proper balancing of the unusually high supply air requirements to maintain positive pressurized clean room spaces with respect to contiguous "dirtier" spaces is critical. Interior surfaces of supply air ductwork as well as the interior building spaces through which this air circulates (both supply and return sides) must be cleaned throughout and at the end of the construction period.

Shop Drawings

Because microelectronics facilities are mechanically and electrically intensive, ⅜" = 1' scale composite drawings and sections should be submitted as formal coordination shop drawings by the contractor to the architect/engineer. This requirement often reveals "value engineering" layout options that may not be fully apparent at the scale of the original documents. The most obvious benefit of these planning documents is the reduction of "after the fact" change order costs resulting from poorly coordinated or installed systems. The time spent in this preplanned coordination will help to prevent significant delays in construction time caused by poor construction field coordination. Many contractors even designate their own mechanical/electrical coordinator on site to orchestrate this intricate coordination of subcontractors.

Facility Start-Up

The final keys to any successful clean room are a well-planned clean room certification period and a systems start-up, or shakedown, period. The certification should be conducted by an experienced clean room testing agency qualified under nationally acceptable standards and preferably independent of any clean room subcontractor. Typically, three certifications are performed:

- The room unoccupied, with no equipment.
- The room unoccupied, with process

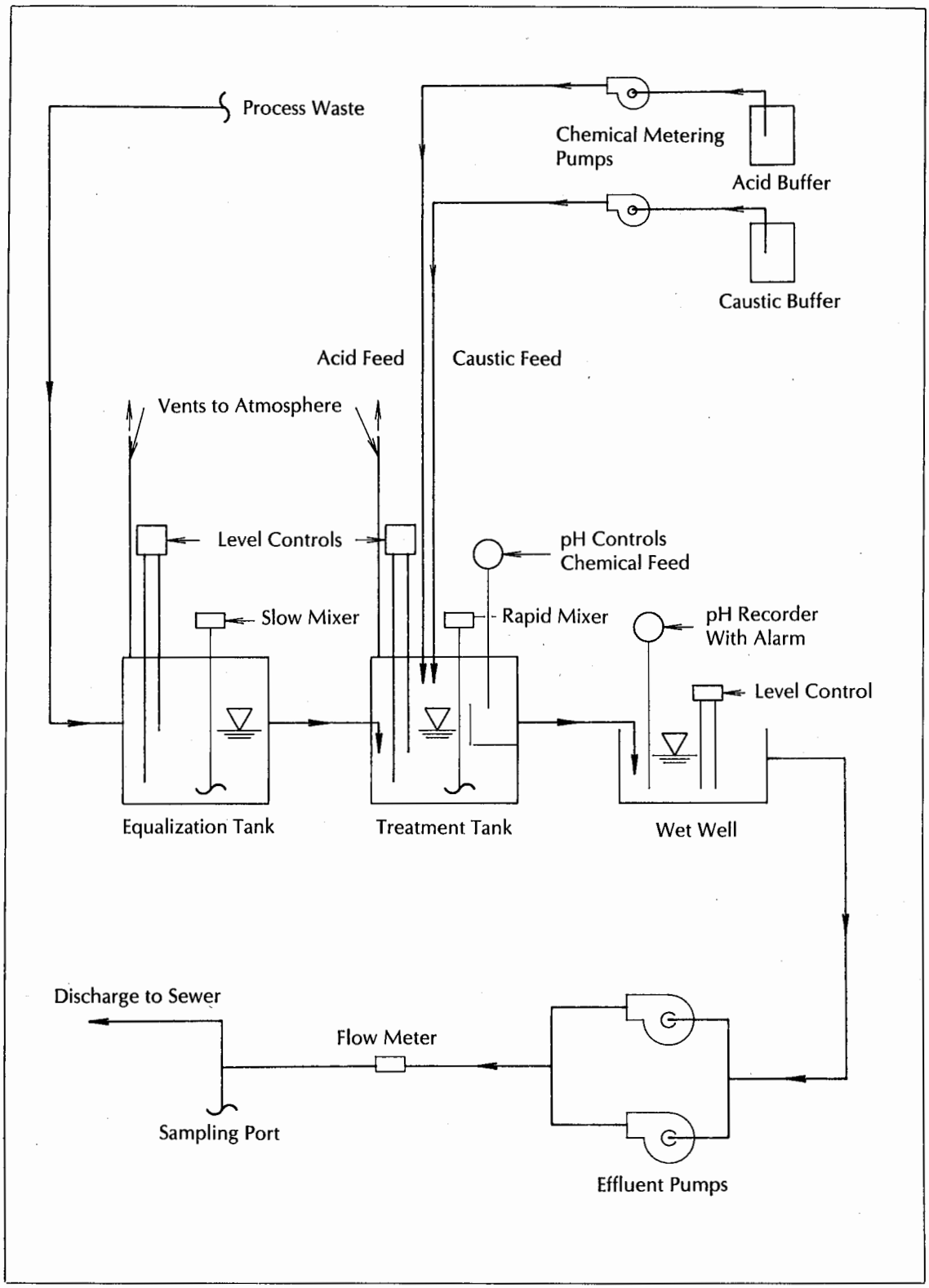


FIGURE 10. A schematic of a typical microelectronics facility waste water treatment system.

equipment in place (at-rest condition).
- The room occupied under normal operating conditions.

Clean room temperature, relative humidity and airflow, as well as other characteristics, will change as equipment and people are introduced into the empty clean room space. Each certification, in its turn, attempts to customize the room's air and control systems, primarily by adjusting supply or return air dampers for balancing air, and by accounting for the various equipment characteristics (for example, added heat load or vibration).

The testing of automatic temperature control systems is also critical at facility start-up. Although state-of-the-art direct digital controls (DDC) are standard equipment, their software can be faulty. Other systems such as process chilled water, high purity water treatment and distribution, process gas distribution, fire alarm, security and fire protection (HALON or wet sprinkler) require similar operational testing. The architect/engineer, owner and contractor must act in concert as a quality control team in order to correctly assess the proper function and operation of all these final systems.

Project Management Issues

In recent years, the planning, design and construction of clean room facilities has evolved to its present advanced state requiring a highly integrated owner, designer and contractor team approach. No single area of the design and construction phases can be overlooked since each phase has critical influence on the final success of the project. However, in the early phases of architect/engineer selection and contractor selection, programming and planning are the key elements.

A multitude of approaches can be used to select an architect/engineer team. As with subcontractors, the most popular method consists of developing prequalified lists of architectural/engineering firms that have had experience in microelectronic facility design and construction. Once this list has been finalized, proposals can be solicited that will identify each firm's approach, unique qualifications, key personnel and any other special considerations. A qualified firm can then be selected and a fee negotiated on the basis of the initial proposal document.

For larger and more complex facility projects, it has become common to include a construction manager as part of the project team. The construction manager is often brought aboard during the early design phases as an active participant in the budgetary analyses and scheduling of the project. Depending on the success of the early project decision-making process, the possibility of issuing early site, foundation and structural steel subcontracts may result in significant time savings. Again, care should be taken to make sure that construction management firms are carefully screened and selected only from qualified firms that are active in the high technology construction market. This selection may be based on proposals received from various construction management firms that identify their approach, team members, qualifications and, if requested, fees for doing both design consultation (generally a lump sum) and construction phase management (generally a percentage of the construction costs). As a rule, the architect/engineer and owner team should be jointly involved in this process.

The final project team is composed of the following typical elements:

- An owner/end user representative
- An owner/facilities representative
- The architectural/engineering design team
- The construction team (construction manager or general contractor, subcontractors)
- Specialty design consultants (process and vibration/acoustical)
- Post-construction owner representatives — clean room and plant managers

A communications system that integrates each of these functional team members is critical for complete project control. In the early programming and planning stages, the professional team will be working closely with user and facility owner groups to incorporate information from client input. It is up to the

professional team to indicate the feasibility of any "wish list" items in light of budget/project goals, and include them in the construction program if possible.

The design and construction phases of the project require constant value engineering and a review/approval process at the end of the various traditional project phases (feasibility/programming, schematic, design development and construction documents). Ongoing reviews of the cost model and schedule must also be performed in order to monitor the status of the target budget and to uncover any schedule constraints.

The development of the final construction cost is the result of competitively bid packages assembled via the contract delivery process. The evaluation and procurement of the various subcontract components of the work should have the involvement of the owner, designer and contractor. This involvement is typical of the construction management process and is not usually part of the traditional general contractor competitive bidding process. Regardless of the ultimate construction delivery process selected, the architect/engineer, owner and contractor must coordinate their efforts during the construction period to accommodate any necessary changes under the requirements of the contract. Unlike a normal building project, changes to microelectronics facilities are frequently the result of unexpected state-of-the-art changes in semiconductor technology, particularly in such sensitive areas as microcontamination control and process equipment advances.

After the design and construction are completed, the critical building system start-up or shakedown process begins. Only through the intensive testing of the building's critical systems — particularly automatic temperature controls, certification of clean room environments, certification testing of process and ultra-pure water piping systems, and HVAC system balancing — can the successful operation of the project's process can be assured. Typical issues of concern to the project team during the design and construction phases from assessment and project scope to identification of documents and project cost modeling are summarized in Table 3.

Particle Control: Operator Discipline

The best operating clean room design can be nullified by operator negligence. Contamination control requires the facility staff to follow disciplined operating procedures. In Japan, sophisticated contamination control management procedures have substantially reduced the degree of outside contaminants entering clean room facilities on the clothing or bodies of the operators. Figure 11 illustrates the control typical in these Japanese facilities from the time the worker arrives at the plant to the time that he or she enters the clean room environment. Such an approach is ultra conservative, yet there is no argument that the disciplined pattern, as well as worker understanding of contamination control, result in higher production success. Clean room managers estimate the percentages of product loss to be as high as 50 percent for some processes due to inadequate environmental management of workers. Inattention to correct clean room entry procedure, even wearing make-up or aftershave, can affect the quality of the day's production or lead to its loss.

Future Trends

The pending revisions to Federal Standard 209B, which will be designated Federal Standard 209C, will attempt to respond to state-of-the-art changes in clean room designs for today and into the future. The major recommended changes in 209B will be the addition and definition of Class 1 and Class 10 clean rooms. The number of sampling points will be specified differently, and a distinction will be drawn between unidirectional and non-unidirectional flow rather than laminar and turbulent flow. Minimum sampling volumes will be set based on the amount of anticipated errors in sample testing. Verification requiring particle sampling will be performed both initially and periodically, and all particle counting instruments will require periodic calibration.

This forthcoming revision of 209B was proposed by the current RP-50 Committee of the Institute of Environmental Sciences (IES), and breaks new ground by including particle sizes smaller than 0.5 microns as well as

TABLE 3

Design Decisions For Determining Project Cost & Success

1. Assess Existing Conditions
   - Geotechnical
   - Environmental (if applicable)
   - Master Planning Impact
   - Permit & Approval Procedures (Macro/Micro)
   - Pedestrian & Vehicular Access
   - Site Sensitivity

2. Agree on Scope of Project
   - Resolution of Facility Square Footage
   - Research & Development Area Requirements
   - Support Area Location & Requirements
   - Establishment of Design Criteria by Specific Spaces
   - A Reliable Utility & Equipment Matrix
   - Agreement on Facility Design Image

3. Prepare a Detailed Design Task Schedule
   - GANTT Chart
   - Critical Path Method Diagram
     - Identify Design Tasks & Approach Milestones
     - Allow for Coordination & Review Times by Architect/Engineer & the Owner
     - Constantly Test & Revise the Schedule as Necessary
     - Check That Each Task Noted Is Within Definable Scope of Work

4. Establish Project Control
   - Perform Budget Testing
   - Develop Accurate & Timely Communications
   - Plan Reliable Reporting Procedures
   - Designate Monthly Corporate-Level Meetings
   - Establish Design Sign-Off Procedures
   - Set Agenda & Other Meeting Standards

5. Identify the "Undeliverable" Documents at Each Phase
   - Agree on the Extent of Written Materials, Particularly Specifications
   - Prepare Mock-Up Drawings at the Start of Each Design Phase
     - Establish schedules & Cost Estimate Formats
     - Allow Formal Presentations of Each Major Submittal
     - Keep Project Team Comments on One Master Set

6. Initiate & Maintain a Project Cost Model
   - Establish a Preliminary Project Cost Model of as Many Components as Practical
   - Develop Timely Comprehensive Estimates to Test the Original Cost Model
   - Make Cost Estimating Important to the Project Function
   - Understand Your Value Design Options That May Not Affect the Actual Program
   - Carefully Assess Contingency Percentages to Be Used
   - Evaluate Local Labor & Tax Conditions & Potential Impacts on the Cost Model of Productivity in the Geotechnical Project Area

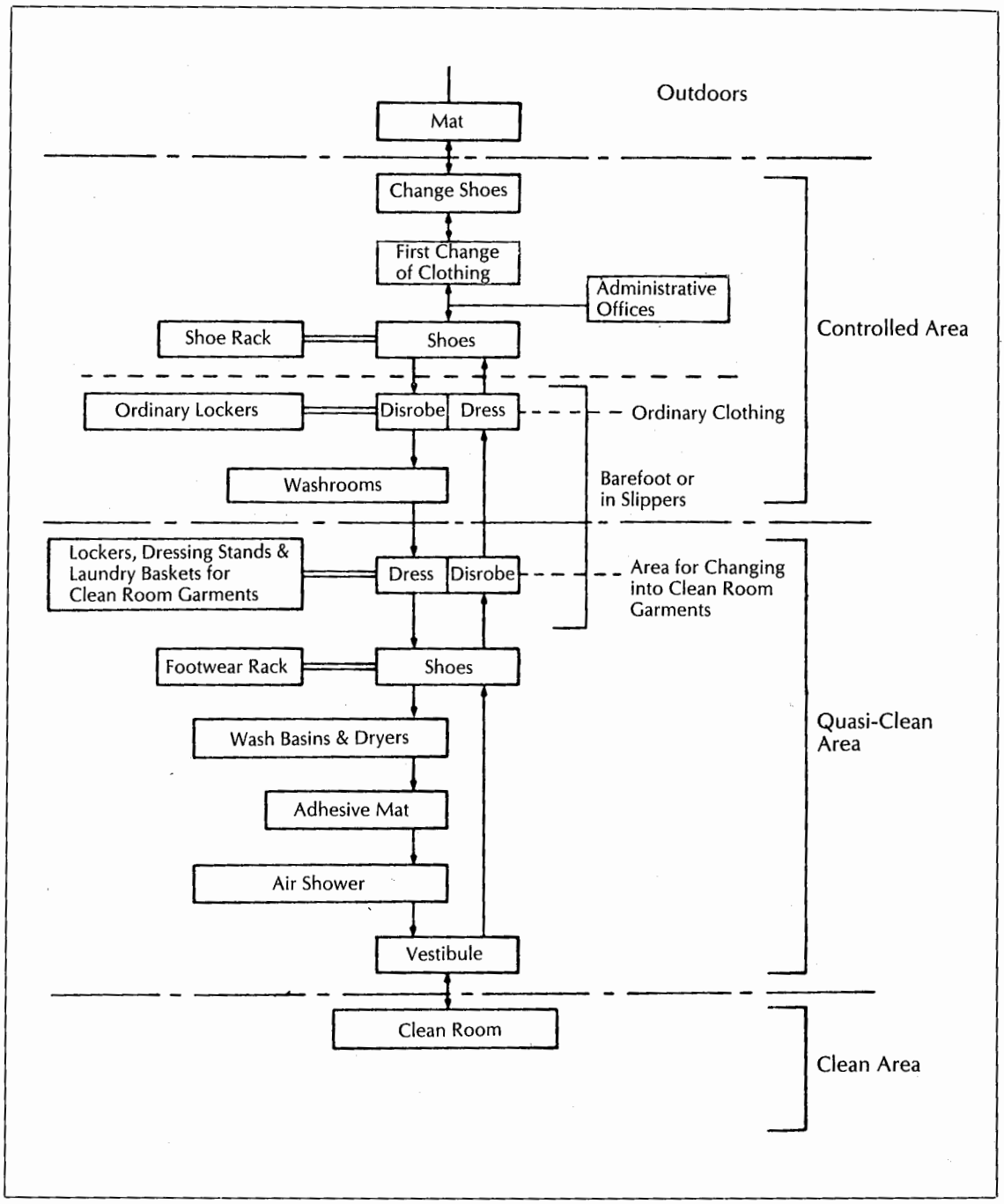


FIGURE 11. A clean room entry diagram.

including Classes 1 and 10. The revision represents the contributions of scores of specialists in the contamination control field who hope that it will improve statistical measurement practices.

These proposed changes will begin to satisfy the needs of the microelectronics industry for cleaner air classes and smaller particle sizes. However, there will continue to be a disparity between the desire for more accurate clean space characterization and the willingness to bear the costs for such char-

acterization.

The physical characterization of clean rooms is also changing. The recent trend has been the prepackaging of the clean room envelope, specifically the use of modular clean room components to construct clean room spaces. This trend is particularly important in light of the rapidly changing semiconductor manufacturing technology that dictates the need for enormous flexibility within these process spaces. The manufacturers of these prepackaged clean rooms point to the exact tolerances achieved during factory assembly. In fact, many of these modular clean rooms are initially assembled at the factory, then disassembled, delivered to the job site and reassembled at the construction site. Such systems are expensive in terms of first capital cost, yet they they do have such advantages as short knock-down capability and short installation time.

Beyond prefabricated systems comes the concept of flexible components such as unistrut assemblies and finished laminated walls that can actually create a reusable field-assembled clean room space. This concept meets the need for flexibility since components of the walls can be taken apart, moved or retrofitted to accommodate equipment changes.

Because sophisticated clean room prices can run between $350 to $750 per square foot in terms of net space cost, the future trends in clean room design will primarily center on the use of automated cassette-to-cassette wafer transfer clean tunnels as a potential replacement for human-occupied clean rooms. These facilities will eliminate the human and equipment contamination factors with robotic process lines that have only critical portions of equipment left in the clean spaces, thereby reducing clean room size. Without the need for wide clean room tunnels on the order of 12 to 14 ft. wide, the first cost in terms of capital installation will be reduced as will the extremely high energy costs per year for air recirculation since these spaces will be smaller and less air will be required. The basic reason for applying these robotic systems is the lessening of both capital and operating costs of tomorrow's clean rooms while providing even cleaner spaces through the elimination of human particle contributions. Laminar flow protective wafer transfer systems will significantly improve the microcontamination control that protects the wafer's surface, thereby increasing production yields at submicron geometry levels.

Conclusion

Since the clean rooms of today generally have an effective operating life of three to five years without technological change, future facilities will require unique flexibility requirements to accommodate anticipated changes in equipment and processes. The elements of design and construction described here represent the types of planning that will allow tomorrow's researchers to make the same type of technology gains in the twenty-first century as we have witnessed in the last four decades.

It is interesting to note that as integrated circuits have become increasingly more miniaturized, so has the production environment, further cutting back the overall cost of this elaborate manufacturing process. The amount of space for these process clean spaces will decrease yet more as the need for smaller, faster and unique microelectronics grows. The success of these facilities will still depend, however, on the carefully planned human element of programmed spaces and proper design.

ACKNOWLEDGEMENT — *The authors wish to thank Stephanie C. Hodal, Director of Communications for Symmes, Maini & McKee Associates, for her assistance in preparing and editing this article.*

WILLIAM L. MAINI, *P.E., is President and CEO of Symmes, Maini and McKee Associates, a multidisciplinary architectural and engineering firm in Cambridge, MA, that specializes in the design of advanced technology facilities. A graduate and former staff member of MIT, Mr. Maini was also one of the firm's founding principals. He remains active in building system design and project admin-*

istration as well as in corporate administration and finance.

MICHAEL K. POWERS, *P.E., is a Principal and Director of Project Management at Symmes, Maini and McKee Associates. A member of ASCE and a senior member of the Institute of Environmental Sciences, he is responsible for overall project administration, cost control, scheduling and design coordination of highly technical projects including semiconductor facilities, research laboratories, data processing centers and commercial complexes. He received his B.S. in Civil Engineering from Northeastern University in 1971.*

MARIO J. LOIACONO, *P.E., is a Principal and Director of Mechanical, Electrical and Engineering Services at Symmes, Maini and McKee Associates. A 1971 graduate of Northeastern University in Mechanical Engineering, he has been involved in mechanical and electrical design for numerous microelectronics and clean room projects for institutions and private industry.*

Truss Bridge Rehabilitation Using Local Resources

Imaginative use of local resources including materials, expertise and labor can result in cost efficient projects.

ABBA G. LICHTENSTEIN

OLD BRIDGES need not die. They can be rehabilitated and put back into service. For many existing bridges where rural conditions still prevail and where traffic has only increased moderately over the years, rehabilitation has particular practical and economic application, especially when the structure has major historic significance.

The Stuyvesant Falls Bridge, in the hamlet of Stuyvesant Falls in Columbia County, New York, is a case in point (see Figure 1). Located on County Road 22, the bridge spans the Kinderhook Creek in the town of Stuyvesant, about 20 miles south of Albany on the east side of the Hudson River. The entire hamlet, including the bridge, is listed in the National Register of Historic Places.

The area now called Stuyvesant Falls was settled before 1667, and in 1800 became the site of the first paper mill in Columbia County. The mill took full advantage of the water power supplied by the Upper Falls that lie approximately 300 feet (100 m) upstream from the bridge. Today, the Niagara Mohawk Power Company operates an hydroelectric station at the Upper Falls. Stuyvesant Falls itself can be best described as a charming hamlet, characterized by large, well-kept wood-frame houses lining County Road 22 from the bridge to the center of Stuyvesant Falls.

The Stuyvesant Falls Bridge

The Stuyvesant Falls Bridge is a single span, steel, two-truss structure erected in 1899 by the Berlin Iron Bridge Company of East Berlin, Connecticut. Columbia County shoulders the maintenance responsibility for the bridge and has performed routine maintenance of sand blasting and painting over the years. The only heavy maintenance on the structure was performed in 1939, when the original wood deck was dismantled and the present open-grid steel deck was installed.

The two steel trusses, measuring 34 feet (10.4 m) top to bottom chord and 19.4 feet (5.9 m) C_L to C_L, carry a 17-foot (5.2 m) wide

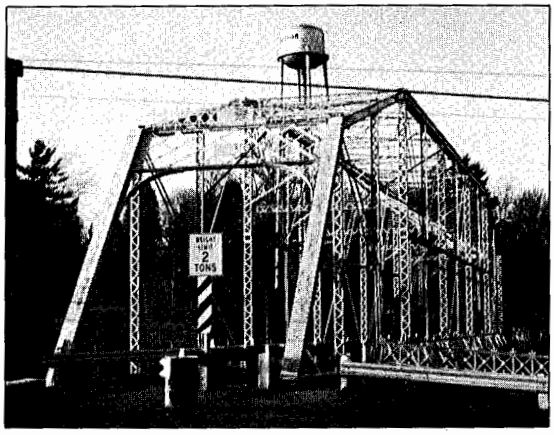


FIGURE 1. The Stuyvesant Falls Bridge viewed from the south.

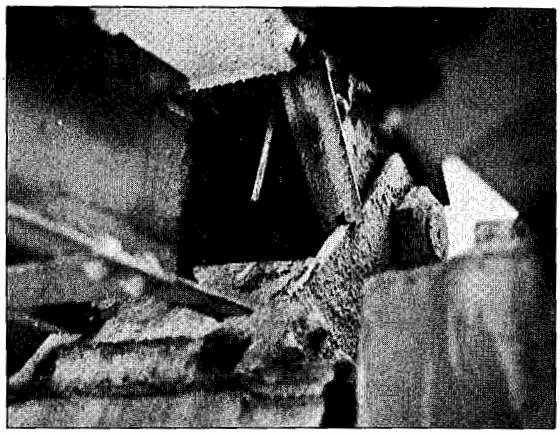


FIGURE 2. A view of the broken lower chord on the east truss (L_0 - L_1).

roadway over the Kinderhook Creek in one simple span 202 feet (61.6 m) long. A 4.7 foot (1.4 m) sidewalk is cantilevered off the upstream truss. The original trusses and floorbeams remained untouched.

An inventory under a state-sponsored bridge inspection program in 1978 revealed that the bridge had structural capacity problems, and recommended that the superstructure be replaced. The estimated construction costs totaled $1.7 million, and did not include engineering or legal costs, or costs for additional right-of-way required for widening the roadway to standard. The state and county were to contribute 20% of this amount based on a specific federal-state funding formula, plus they were to purchase the needed properties for the right-of-way.

After reviewing these costs, the Columbia County Highway Department decided to investigate the feasibility of the county, on its own, making the required repairs and upgrading the bridge. A consulting engineer was engaged to perform this engineering study. An in-depth inspection of the bridge by the consultant then revealed the following structural deficiencies:

1. The bottom chord was entirely severed by corrosion at the south bearing, and the inclined truss member and shoe had moved outward longitudinally 9 inches, and the rocker had twisted. See Figure 2.

2. The connections of three vertical truss members to the floorbeams at the panel points were corroded to the point of uselessness. See Figure 3.

3. Many roadway stringers were rusted and cracked in the end panels at the abutments where salt and dirt had accumulated around them.

4. All floorbeams were in good condition, but could only carry a 7 ton live-load safely.

5. The trusses rated well, except for 16 subpanel verticals whose rating was 9 tons.

6. Other deficiencies requiring rehabilitation included the twisted rocker, deteriorated sidewalk and railing areas, and general deterioration of the paint protection over the entire bridge.

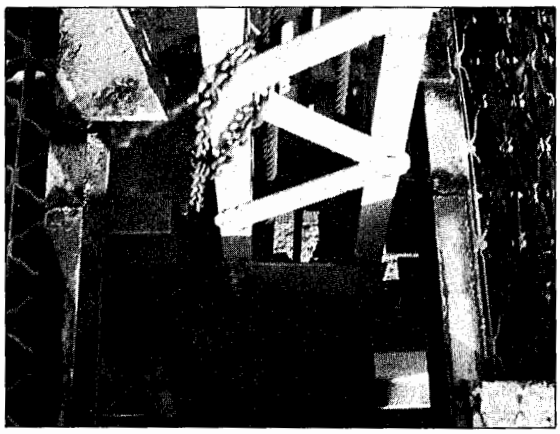


FIGURE 3. Broken vertical on the east truss (L_{11} - O_{11}).

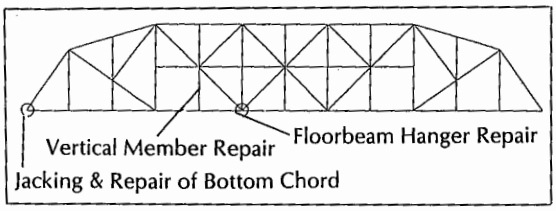


FIGURE 4. Elevated view of the east truss noting repair locations.

Following the consultant's inspection, the county decided to effect immediate temporary repairs for deficiencies in the broken bottom chord and in the corroded connections of three vertical truss members. The County Maintenance Department, under direct instruction of the consultant, first placed wire ropes tensioned by turnbuckles between panel points and load posted the bridge for 2 tons. With this expedient approach the bridge could then be used to some extent while the county and the consultant studied alternative methods for upgrading the bridge.

Repairing the Bridge

Further study by the consultant revealed that economically correcting the bridge's deficiencies enumerated above was feasible. While the necessary repairs, *per se,* were eligible for federal funding assistance, the substandard 17-foot width of the roadway posed a grave obstacle to gaining initial approval for the project and made the chances for ultimate project approval virtually nil.

Since upgrading the roadway to standard would have negated the county's intention to undertake the project itself, and do it economically, the county was left with two options. The first alternative was to solicit bids from contractors to effect the repairs. The second alternative was to have the relatively inexperienced but enthusiastic county maintenance personnel perform the work, while renting necessary equipment and buying structural steel as required.

The county, following consultation between the highway superintendent, his foreman and the consultant, opted for the second alternative. The consultant subsequently discovered that the county possessed a stockpile of structural steel that had been stored over the years at its yard. An inventory of the steel was performed and the needed temporary or permanent steel sections were used whenever possible from this inventory, with members chosen that met or exceeded minimum calculated requirements.

In order to begin repairs, the first order of business was to jack up the south end of the trusses approximately one foot (0.3 m) and remove the load from the twisted shoe (see Figure 5). Once the jacking was completed, the repair could be effected. The truss member and rocker were pushed horizontally by another jack to a calculated point and a chord mending device was installed to keep them there permanently. The jacks and auxiliary beams were then removed. The verticals and floorbeams were reconnected with permanent-type details, and the sub-verticals of the two trusses were reinforced with special "horseshoe"-shaped collars (see Figure 6).

The next repair in line was the strengthening of the floorbeams. Without removing the grating (which was in fair condition) and stringers, a platform was erected under each floorbeam (see Figure 7). Four angles were bolted onto the webs of the beam as close to the flanges as possible. The corroded stringers

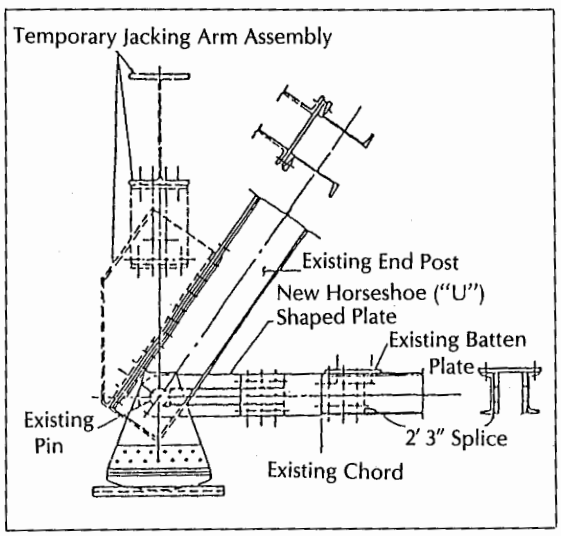


FIGURE 5. Jacking details and repair of the bottom chord.

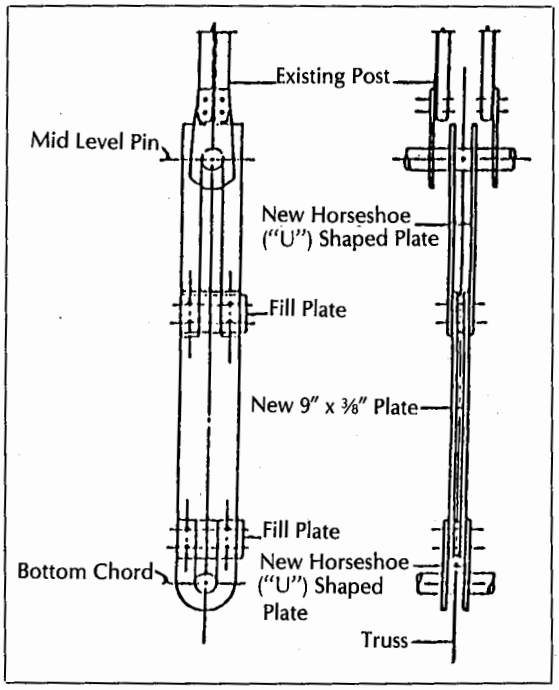


FIGURE 6. Vertical member repair.

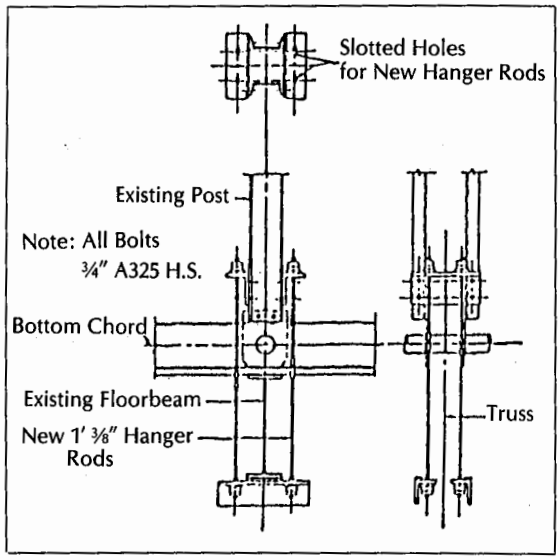


FIGURE 7. Floorbeam hangers.

were then left in place, but beams of similar depth were set alongside the bad ones and shimmed into place. The railing and sidewalks were repaired in spots where needed. Since cleaning and painting were not priority tasks, they were deferred to the following season. The bridge was reopened to traffic with a capacity of 15 tons — adequate to handle all traffic including buses, fire engines, ambulances and other community vehicles. For the occasional runaway heavy truck, the bridge's operating rating is 20 tons.

With an inexperienced work crew, the time necessary to complete the project was four months. No time was lost for permit application or outside agency approvals. The cost of the project to the county was $59,000 in addition to the salaries of the maintenance workers and the structural steel salvaged from the yard. In all, the county minimized its costs, preserved an historic structure, and gained a maintenance workforce that had acquired additional skills.

The county solved the Stuyvesant Falls Bridge problem by doing most of the work itself, and derived a great deal of satisfaction on account of this reduced expenditure. The then Governor of New York, Hugh Carey, recognized the Columbia County Superintendent of Highways, Richard Brady, with a special award for his imagination and good sense in restoring that historic bridge. Another plaque in similar recognition was given to Richard Brady and Columbia County by the National Association of Counties.

The restoration of the Stuyvesant Falls Bridge provides a surprising and encouraging example of how resourceful engineering can contribute, cost effectively, not only to the preservation of an historic landmark, but also to the necessary upgrading of a modern roadway system.

ABBA G. LICHTENSTEIN, *P.E., is Chief Engineer of A.G. Lichtenstein & Associates, Inc., a consulting engineering firm specializing in designing bridges and dams, and evaluating existing bridges. Since 1965, Lichtenstein's company has inspected and rated nearly 3,500 bridges, including various fixed and movable structures for highways, railways and pedestrians. He has taught seminars at Cornell, Ohio State, Hofstra, Rutgers, and the Universities of Missouri, Pennsylvania and Wisconsin. He received the Honorary Degree of Doctor of Engineering from Ohio State University. His firm performed the consulting services for the Stuyvesant Falls Bridge.*

Back Bay Boston, Part II: Groundwater Levels

Man-made structures that permanently lower groundwater levels can have adverse effects on buildings with water table sensitive foundations.

HARL P. ALDRICH, JR. &
JAMES R. LAMBRECHTS

THE TEMPORARY or permanent lowering of the groundwater table can adversely affect both natural and constructed environments, causing ground subsidence, flooding and damage to structures. The Back Bay section of Boston serves as an excellent site for the study of the causes and effects of groundwater level diminishment, and provides ample reasons for the need to monitor and maintain groundwater levels to preserve building foundations.

The second in a series of studies on Back Bay, this article summarizes groundwater levels in Back Bay since the area was filled more than 100 years ago, and traces the effects of construction of sewers and drains, subways and other transportation corridors, and buildings on the groundwater table. Part I described the geology of Back Bay as well as subsurface soil conditions and the topographic development of the area, concluding with a discussion of building foundation practice through the turn of the century, a practice based primarily on untreated wood piles.[1] Part III, now in preparation, will complete the series, documenting foundation design and construction practice from 1900 to the present.

Background

This study focused on the geographical area bounded by the Massachusetts Bay Transportation Authority's Southwest Corridor Project (south), Charles Street (east), Massachusetts Avenue (west) and the Charles River Basin (north). This area currently encompasses the Back Bay Historic District (primarily between Boylston Street and Beacon Street) and the central spine across Back Bay where major projects have been constructed during the past 30 years. The South End neighborhood, located to the south of the Southwest Corridor, was excluded, primarily because little data on groundwater levels exist for this area.

During the nineteenth century, a tidal estuary of the Charles River known to Boston residents as the Back Bay (see Figure 1) was filled to create land for an expanding population. Most of the homes, churches and other buildings constructed prior to 1900 were founded on wood piles driven through fill materials and organic soils to bear in the underlying sand and gravel or clay stratum. For the most part, the tops of these piles were

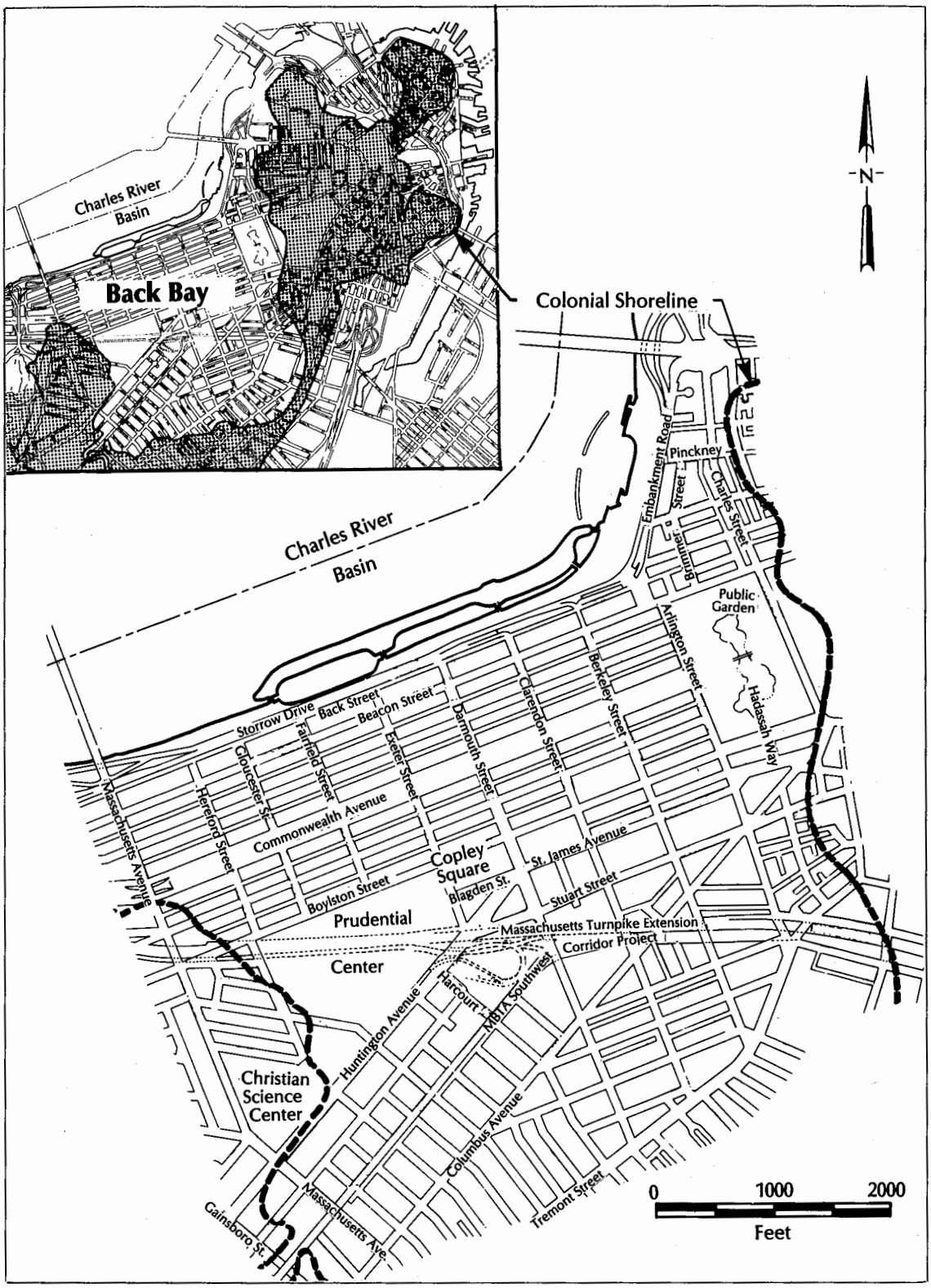


FIGURE 1. A map of Back Bay Boston showing the location of the colonial coastline.

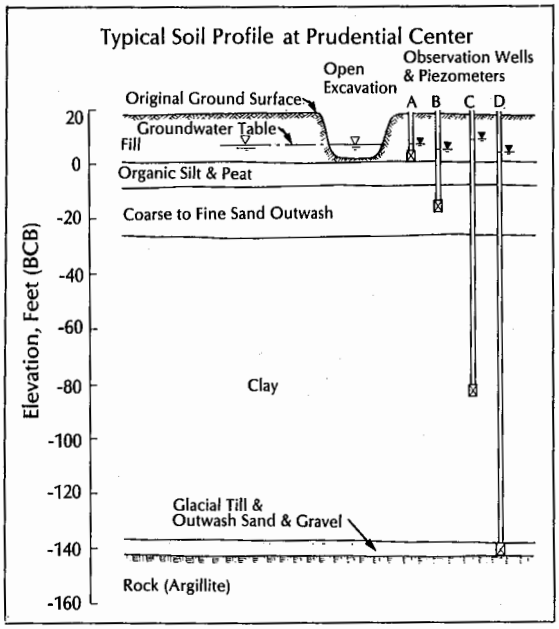


FIGURE 2. A typical soil profile, with groundwater table. Well A shows the static water level.

cut off below the water table at the time of construction with the expectation that they would be preserved if permanently immersed below the groundwater table.

With construction of sewers, drains, subways and the basements of buildings below the water table, some of which leak, the groundwater level has dropped in Back Bay. Where wood piles have been exposed to air for some time, the piles have rotted when attacked by fungi, borers and other organisms. A few buildings have settled and cracked, requiring owners to underpin their structures at great cost in order to restore the foundations.

The Groundwater Table

The Webster dictionary defines the groundwater table as the level below which the ground is saturated with water. In geotechnical engineering, it is the stabilized static water level in an open excavation, or in a shallow well or piezometer, as illustrated by well A in Figure 2. In Back Bay, the water table generally occurs within the fill stratum from 10 to 15 ft. below the ground surface.

Three principal water bearing aquifers occur in Back Bay, separated by impervious soils. The lowest aquifer, a relatively thin, but apparently continuous stratum of outwash sand and gravel or glacial till underlying the Boston blue clay, is relatively pervious. The middle aquifer is a compact gravelly sand stratum up to 20 ft. in thickness confined between the blue clay and a near continuous stratum of organic silt and peat. This pervious outwash material occurs primarily over the western and northern sections of Back Bay. It does not exist in the Copley Square area. The top aquifer is the artificial fill, commonly a silty coarse to fine sand, placed during the nineteenth century. The groundwater level in the fill, the top aquifer, is the principal concern in Back Bay.

In the westerly section of Back Bay, where the sand outwash stratum occurs below the relatively impervious organic soils, a second "water table" may be present — one that may differ from the water table in the fill. This situation is represented by well B in Figure 2. If the water level in all wells or piezometers in the figures were equal, then the groundwater would be hydrostatic with depth.

"Normal" Groundwater Levels. If there were no loss of groundwater by pumping and leakage into sewers and drains, and no additions to the groundwater from leaking water mains and other sources, the probable "normal" water table in Back Bay would be as shown in Table 1.

In colonial times, Back Bay was a tidal estuary that had a mean water level approximating the mean tide in Boston Harbor (el. 5.65 Boston City Base[2]). Following the completion of the Mill Dam along Beacon Street across Back Bay in 1821, and until 1880 when most of Back Bay was filled, water levels in the receiving basin east of Massachusetts Avenue were variable and generally below mean tide.

After Back Bay was filled, the groundwater table would have been expected to rise above mean tide level. The land mass was bounded on the north by the Charles River and on the south by South Boston Bay, both tidal. Surface water from rainfall and snowmelt that percolated into the ground would have been expected to raise the water table until a

TABLE 1

Probable "Normal" Groundwater Levels

Time Period	Probable Average Groundwater Level	Comments
Pre-1800; before Back Bay was filled	El. 5.7 (BCB) (mean tide level)	Back Bay was a tidal estuary of the Charles River
1880-1910; after filling	El. 5.8 at Charles River; el. 8± interior	Above mean tide level due to infiltration of rainfall & snowmelt
1910-present; after Charles River Dam completed	El. 8 at Charles River; el. 9.5± interior	Charles River Basin maintained at el. 8.0

Note: Assumes no loss of groundwater by pumping or by leakage into sewers, drains, foundations or basements; and no additions to groundwater from leaking water mains and other man-made sources.

horizontal gradient in the fill was established to conduct groundwater by seepage toward the adjacent bodies of open water. In the latter part of the nineteenth century, the groundwater level in Back Bay was, in fact, approximately el. 8.0 ft.[3]

The construction of the Charles River Dam in 1910 raised the mean water level in the Charles River Basin to el. 8.0. The Back Bay groundwater table would have then been expected to rise further, perhaps to el. 9.0 or higher along Boylston Street. Normal groundwater levels in the area between Charles Street and Storrow Drive would have then been from el. 10.0 down to 8.5, as a result of groundwater runoff from the west side of Beacon Hill.

Major sources of groundwater in Back Bay are infiltration of rainfall and snowmelt, leakage from water mains, and recharge from man-made groundwater recharge systems. The sand outwash receives water from the fill by seepage downward through the organic soil and by direct flow from the fill through holes, trenches and other man-made "openings" excavated through the organic stratum.

Only a fraction of the annual precipitation actually enters the ground because more than 80 percent of the Back Bay is covered by impervious surfaces such as streets, sidewalks and buildings. Even in open, unpaved areas, only part of the precipitation enters the ground. Although most of the precipitation in Back Bay becomes runoff and is carried away by storm drains and sewers, the seasonal variations in the type, and level, of precipitation cause an annual fluctuation in groundwater levels up to about 2 ft. in some areas.

The Charles River may become a source of groundwater in the Back Bay when the water table falls. However, seepage through the fill is severely impeded by remnants of the Mill Dam and the West Side Interceptor along Beacon Street, and by the Boston Marginal Conduit under Storrow Drive. Because the river level is maintained at el. +7.5 to +8.0, its effect on groundwater levels is essentially constant. The river's influence on water levels in the fill decreases rapidly with distance from the river.

The relatively pervious sand outwash stratum also underlies the Charles River. The Mill Dam and Boston Marginal Conduit would not impede recharging in this stratum. However, since the river bottom is also blanketed by organic soils, its influence on piezometric water levels in the Back Bay outwash is uncertain.

Leaky pipes, particularly water mains, can be significant localized sources of groundwater. Cotton and Delaney provided groundwater contours that indicated several mounds where water levels were as much as 5 to 10 ft. above surrounding areas.[4] The overall contribution to the water table from leaking water mains may be about equal to that from precipitation. Cotton and Delaney reported that Boston Water Department data from the early 1940s indicated that water main leakage would have provided an equivalent recharge of 0.73 million gallons per day (gpd) per square mile. This amount is approximately equal to the recharge from 50 in. of precipitation per year, assuming a 30 percent infiltration rate. Storm and sanitary sewers located above the groundwater table can also leak and contribute to groundwater. Because they are not under pressure, their effect is probably minor.

In several areas, permanent recharge systems have been installed to help maintain groundwater levels. Notable examples are the recharge systems at Copley Square and Trinity Church. In these systems, surface drainage from precipitation is collected and directed to drywells or reverse drains. Water then seeps back into the ground through special piping systems. Temporary recharge systems have been used in areas adjacent to construction projects, notably the Prudential Center, to prevent or correct lowered groundwater levels caused by deep excavations and construction dewatering.

Loss of groundwater, and the resulting lowered water levels in Back Bay, occur primarily from leakage into sewers and drains, leakage through walls and floors of subway tunnels, underpasses, building foundations and other structures below the water table, and by pumping from sumps. In addition, water levels may be lowered temporarily by pumping from excavations in order to facilitate construction.

Adverse Effects of Lowered Groundwater Levels

Temporary or permanent lowering of the groundwater table from man-made or natural causes have been shown to adversely affect buildings, streets, underground utilities and other structures, as discussed by Aldrich.[5] Potential problems applicable to the Back Bay are illustrated in Figure 3 and include:

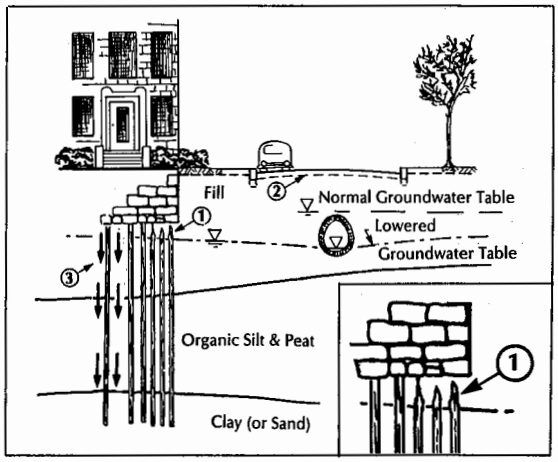


FIGURE 3. Principal adverse effects of lowered groundwater levels: (1) decay of wood piles when exposed to oxygen; (2) ground subsidence due to compression of organic silt and peat; and (3) negative friction (drag) on piles when the ground settles.

- Deterioration of wood piles
- Ground subsidence
- Negative friction (drag) on piles

Deterioration or decay of wood piles is clearly the most serious potential problem associated with lowered water levels. As long as the water table remains above the tops of the piles, and the wood and surrounding soil remain saturated, the wood will not rot. Under these conditions, untreated wood piles can be considered to be permanent.

However, if the groundwater level drops below the tops of the piles, favorable conditions may be present for plant growth and insect attack. A greatly increased supply of oxygen, combined with moisture and moderate temperatures, facilitate the growth of fungi. Grubs or wood borers, termites and other insects may also attack the "exposed" wood.

The butts of piles that are surrounded by fill, in particular sand and gravel as well as ashes and cinders, are more prone to rotting

than are piles that are embedded in organic silt, peat and other relatively impervious soils. When the water table drops, the fine-grained soils remain saturated for a time, thus protecting the piles from immediate deterioration.

The time required for significant deterioration to occur, following a drop in groundwater level below the tops of wood piles, is highly variable. It depends on the species of wood, the type of soil in which the piles are embedded, the amount of moisture, temperature and other factors. Exposure for a few months is not considered serious. However, serious deterioration will probably occur after a drawdown period of 3 to 10 years.

Ground Subsidence. When the groundwater level is lowered, the effective stress on soils that occur below the water table is increased. Buoyancy in the zone of drawdown is lost. If underlying soils are compressible organic soils or soft clays, these materials will consolidate as the soil structure adjusts to the increase in the overburden load. Settlement will also occur if the upper soils dry out and shrink when the water table is lowered.

Most areas of the Back Bay have experienced one or more significant temporary groundwater drawdowns for the construction of sewers and drains, subways, foundations for buildings, and other excavations that have required pumping. For this reason, ground subsidence due to future temporary or nominal permanent lowering of the water table is not considered to be a serious concern.

Negative Friction. All buildings in Back Bay that are supported by piles driven through fill and organic soils — whether they are wood piles bearing in the sand and gravel outwash or marine clay, or are long piles driven to bear in the glacial till or bedrock — will experience negative friction or drag loads when the ground surrounding the piles settles. The building may settle as a result. The potential adverse effects are most pronounced for wood piles that derive their support by skin friction in the marine clay.

While significant negative friction undoubtedly developed in the nineteenth century from the compression of organic soils under the weight of overlying fill, and from temporary groundwater drawdowns, this factor is not likely to be a serious concern in the future.

Construction in Back Bay

The construction and maintenance of embankments, sewers and drains, transportation corridors and buildings throughout Back Bay have affected groundwater levels. The impact of this complex interconnected underground system, shown in Figure 4, on the water table cannot be appreciated without some knowledge of each component.

Mill Dam. The first significant filling in Back Bay took place in 1820 when the Mill Dam was constructed along Beacon Street from Charles Street to Sewall's Point in Brookline, near the present Kenmore Square. From a description given by Howe, a cross section of the dam can be developed as shown in Figure 5 .

As a dam, the structure was relatively impervious to the flow of water from one side to the other, except where it has been breached locally by construction in the past 100 years. However, in a longitudinal direction along Beacon Street, the structure is probably very pervious.

Filling of Back Bay began in 1858 at the Public Garden, continuing westward to Massachusetts Avenue by 1880. From 10 to 20 ft. of sand and gravel fill were placed over soft organic soils that were underlain by a deep clay stratum. Considerable ground subsidence occurred over a long period of time from the compression of the organic soils and, to some extent, the clay.

Concurrent with the filling, a sea wall was constructed along the Charles River to create Back Street that parallels Beacon Street on the water side. The top of this wall is clearly visible from Storrow Drive. A similar wall was constructed in about 1865 behind homes on the water side of Brimmer Street on Beacon Hill. Both walls were composed of dry-laid granite placed on a timber platform and supported on wood piles (see Figure 6). It is probable the walls were ballasted with stone or gravel, similar to the Mill Dam walls.

Construction of buildings followed closely behind the Back Bay filling. All of these

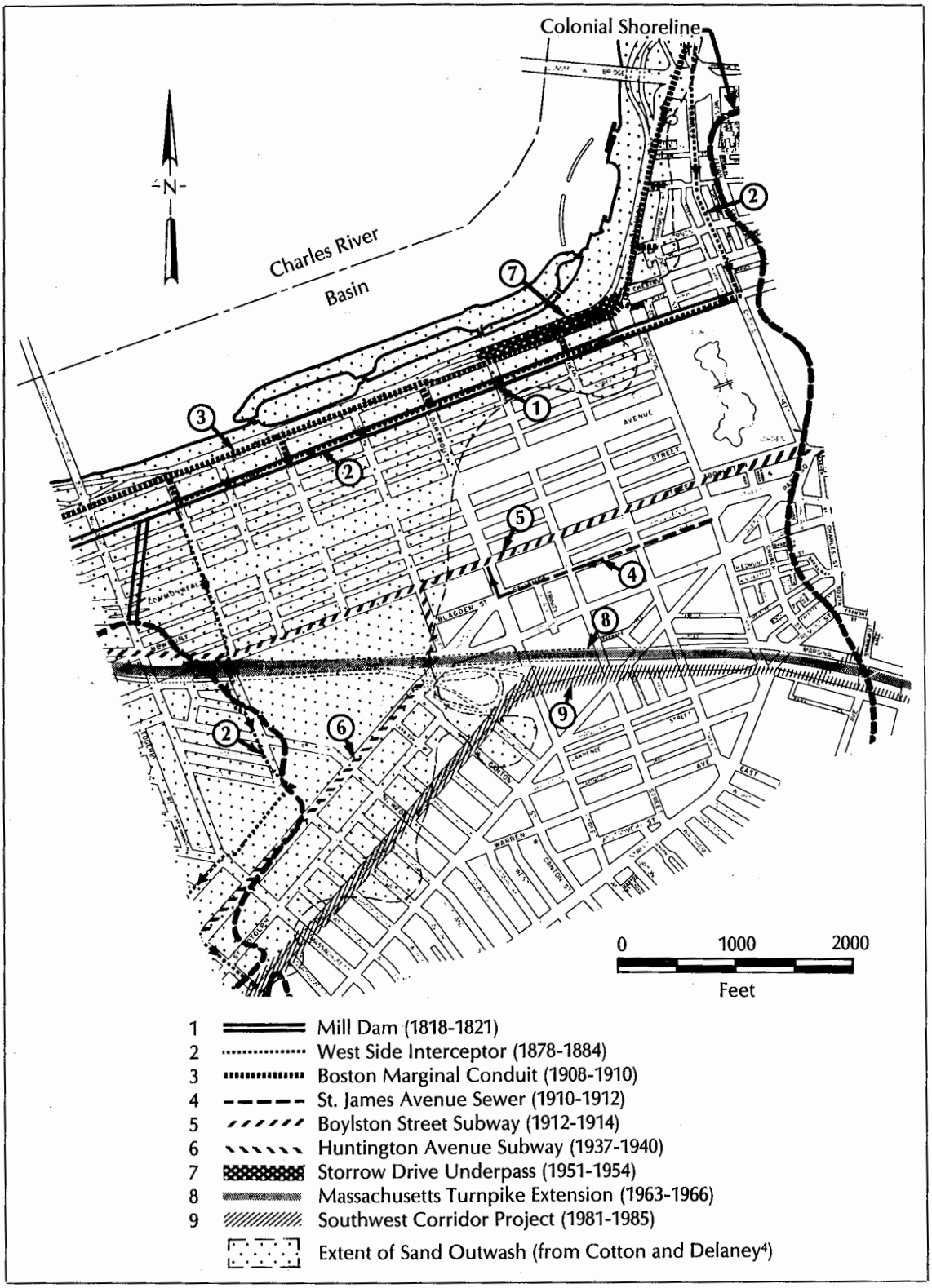


FIGURE 4. The locations of sewers and drains, and major transportation alignments.

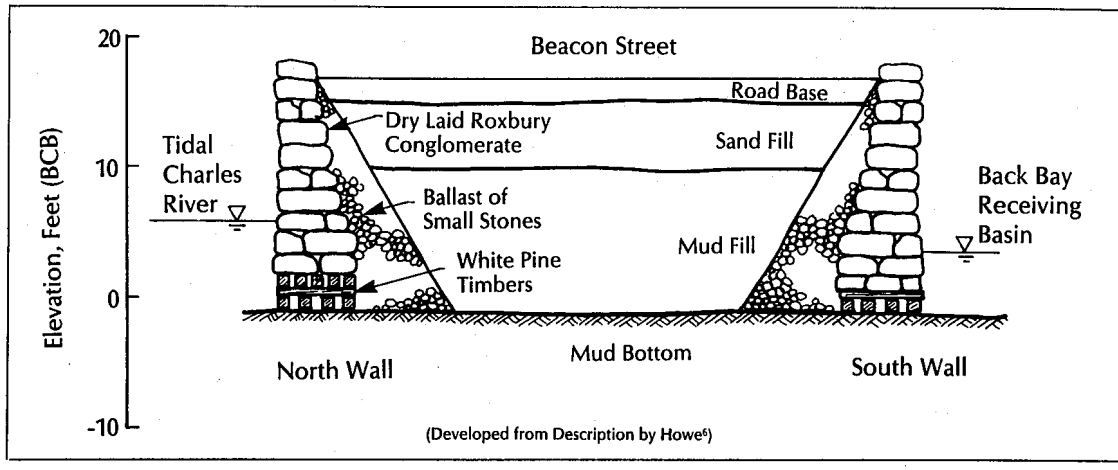


FIGURE 5. A typical cross section of the Mill Dam.

buildings were founded on untreated wood piles cut off typically at el. 5.0, approximately 2 to 3 ft. below the groundwater table.[1]

Sewers and drains in Back Bay have contributed to at least localized depressions in the groundwater table. Furthermore, dewatering for sewer construction undoubtedly caused extensive temporary lowering of the water table in some areas. Plans of the principal existing sewers and conduits in the Back Bay are shown in figures in a report by Camp, Dresser & McKee.[7]

The earliest sewers and drains in Boston discharged by gravity from the hills to adjacent tidal areas. Flow velocities were high and there were few problems. With the development of the low filled-land areas like the Back Bay, the extension of the sewer system created serious drainage problems in Back Bay because of the area's flat gradients and ground settlement.

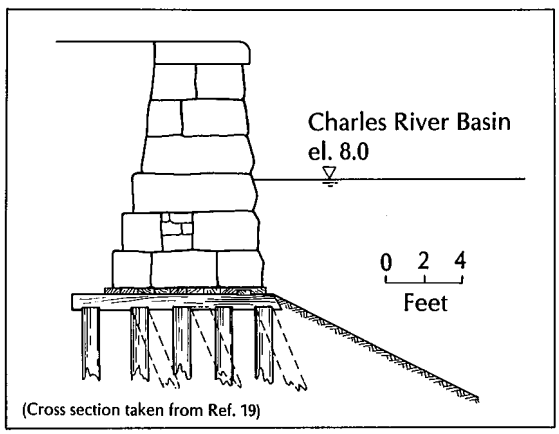


FIGURE 6. The sea wall along Back Street.

Most house drains and sewers were below basement level, and when minimum slopes to street sewers and interceptors were provided, the outfalls were rarely above low tide. As a result, the contents of the sewers were dammed up by the tide during the greater part of every day. (Tide gates were commonly adopted to prevent salt water from flooding the lower reaches of the sewers.) Settlement of the filled land caused numerous breaks in sewer connections and reversals of slope. Deposits of sludge and debris within the sewers and in tidal areas accumulated rapidly, with their attendant health and odor problems.

By 1868, the State Board of Health recognized a serious public health problem and, in 1875, the City Council authorized the Mayor to appoint a commission to study the sewage system and to plan for future needs of the city. The plan adopted became the Boston Main Drainage System.

The Boston Main Drainage System, was constructed from 1877 to 1884. The principal feature of these works was a system of intercepting sewers along the margins of the city to receive the flow from the already existing sewers. These intercepting sewers drained to a pumping station located at Old Harbor Point on Dorchester Bay (Calf Pasture on Columbia Point) where sewage was pumped to Moon

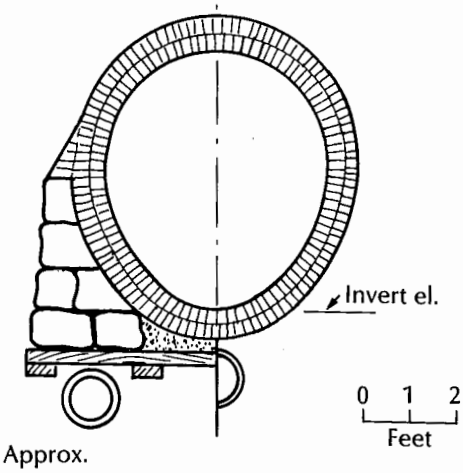
Island and discharged into Boston Harbor on outgoing tides.

Existing combined sewers (storm water and domestic sewage) in the northerly section of the Back Bay that formerly discharged into the Charles River at Beaver, Berkeley, Dartmouth, Fairfield and Hereford Streets, were connected to the West Side Interceptor that was constructed along Beacon Street. Other sewers located south of the railroads drained into the East Side Interceptor that follows Albany Street.

Design and construction of the West Side Interceptor is of particular interest (see Figure 4). It travels down Charles Street to Beacon Street, where it turns westerly down Beacon to Hereford Street, then turns southerly down Hereford and Dalton Streets to Falmouth Street, and then westerly to Gainsborough Street. In the Beacon Street area, the invert grade varies from approximately el. 0 at Beacon and Arlington Streets, to el. -2.4 at Beacon and Hereford Streets and to el. -4.7 at Huntington Avenue and Gainsborough Street.

Excavation and dewatering for the construction would have been required to at least 2 ft. below these grades, into the fill and organic soil on Beacon Street; and to approximately el. -6.0 in the sand outwash stratum in Dalton and Falmouth Streets. So, over 100 years ago, if not before, the outwash stratum experienced its first significant temporary drawdown. Significant ground subsidence and negative friction on wood piles undoubtedly occurred.

The intercepting sewers and the main sewer, from the upper reaches to the pumping station at Calf Pasture, varied in size from 3 to 10.5 ft. in diameter. The larger ones were circular and the smaller ones were generally egg-shaped. The West Side Interceptor was egg-shaped, 57 in. wide and 66 in. high (see Figure 7). Sewers were constructed with double or triple rows of mortared brick, and where piles were required, a timber platform was constructed and the sewer was cradled on mortared granite masonry. It is of considerable importance to note that the intercepting sewers were constructed with an underdrain pipe varying from 8 to 12 in. in diameter that was placed below the sewer to control groundwater during construction.



Approx. Invert el.	Location
0.7	Charles St. at Pinckney St.
0.4	Charles St. & Beacon St.
-0.6	Beacon St. at Berkeley St.
-1.2	Beacon St. at Dartmouth St.
-2.4	Beacon St. & Hereford St.
-3.0	Hereford St. at Boylston St.
-3.9	Falmouth St. at Mass Ave.
-4.7	Gainsborough St. at Huntington Ave.

Elevations are from Wastewater System Maps prepared by Metcalf & Eddy, Inc., for the Boston Water and Sewer Commission.

FIGURE 7. A typical cross section of the West Side Interceptor. Invert elevations at selected locations are noted.

Observation wells in Back Bay at that time indicated water levels similar to those measured before sewer construction, but within 10 years, in 1894, areas were found where the groundwater was as low as el. 5 or lower, indicating that there was leakage into low-level sewers or that groundwater was being pumped.

The Boston Main Drainage System was designed with sufficient capacity to carry the estimated dry weather flow of sanitary sewage and a small volume of storm water. Excess storm flow and diluted sewage from the West Side Interceptor were discharged into the Charles River at numerous overflow outlets.

Boston Marginal Conduit. With the construction of expensive homes along the Charles River, there were increasing demands to eliminate the odors and nuisance of the tidal basin. Under the Acts of 1903, a half-tide dam

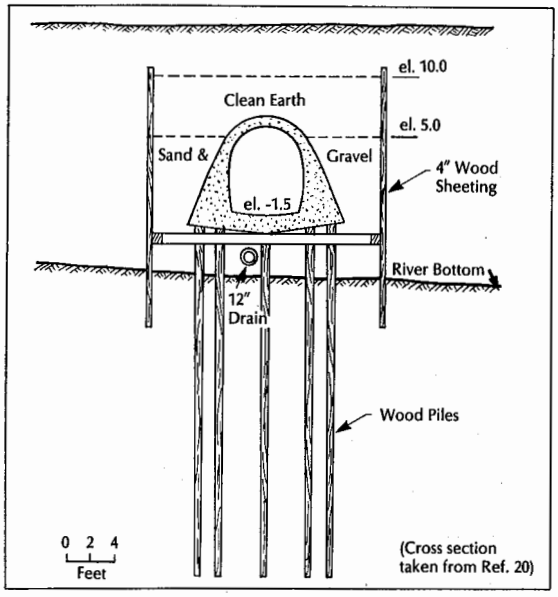


FIGURE 8. A typical cross section of the Boston Marginal Conduit.

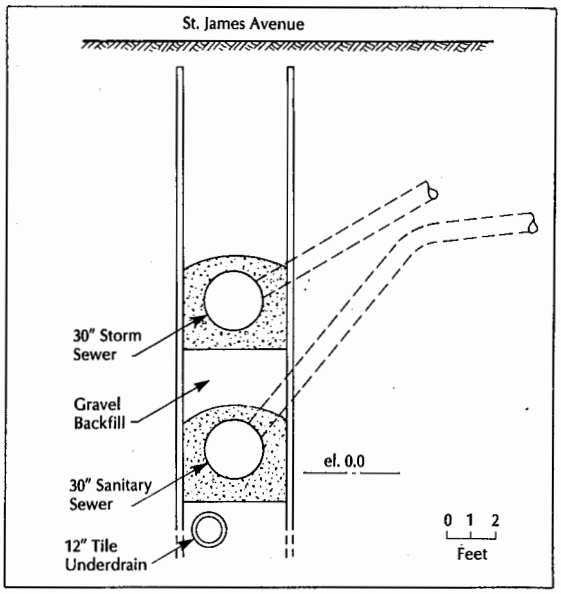


FIGURE 9. A typical cross section of the St. James Avenue sewer.

was completed in 1910 at the location of the former Craigie's bridge, where the Museum of Science is now located. The dam was constructed with gates and a lock to maintain the water level in the Charles River basin at approximately el. 8.0.

As part of the dam project, the Boston Marginal Conduit was constructed along the Boston side of the basin to collect flows from Stony Brook, and mixed sewage and storm water overflows from the West Side Interceptor that formerly discharged into the river at the sea wall (see Figure 4). Water was to be maintained at a low level in the conduit by means of tide gates constructed at the outfall below the Charles River dam.

The marginal conduit was constructed in a 100-ft. wide earth fill embankment placed immediately north of Back Street, beyond the old dry rubble retaining wall. Presently, the conduit lies beneath Storrow Drive. Over most of its length, it is a reinforced concrete horseshoe-shaped section 76-in. wide by 92-in. high, supported on wood piles, as shown in Figure 8.

The structure was constructed level with an invert grade estimated at el. -1.5. Drawings indicated that it was built within a double row of tongue and groove wood sheeting that was driven into the organic silt and left in place. Again, a large diameter underdrain pipe was placed just below the marginal conduit to facilitate dewatering during construction.

When the Storrow Drive underpass was built in 1951, a portion of the conduit was relocated inland, away from the river. The relocated section, from Dartmouth Street to Mt. Vernon Street, was an 8-ft. diameter reinforced concrete pipe with an invert grade at el. -1.5. An underdrain pipe was placed beneath this new pipe, and it was apparently connected to the old underdrain when the relocated section was tied in.

The Mill Dam, West Side Interceptor and the Boston Marginal Conduit act as dams impeding the flow of groundwater from the Charles River basin into the Back Bay. Furthermore, while relatively impervious perpendicular to their axes, they can conduct groundwater with relative ease in a longitudinal direction.

The present system of low level sewers was constructed throughout Back Bay between 1910 and 1912. Underdrain pipes again were commonly used as shown in Figure 9, which presents a section through the St. James Avenue storm and sanitary sewers. By that time, nearly 75 years ago, there was little

doubt that groundwater leaked into sewers, that the problem was widespread and that groundwater levels in Back Bay were controlled primarily by this leakage.

Two major subways have been constructed across Back Bay by the Boston Transit Commission (now known as the Massachusetts Bay Transportation Authority). Between 1912 and 1914, the Boylston Street subway tunnel was built, and from 1937 to 1940 the Huntington Avenue subway was added. Their locations are shown on Figure 4 on page 37.

The Boylston Street subway crosses Back Bay from Massachusetts Avenue to Charles Street. Within this area, the bottom of the subway varies from approximately el. +3.0 at Massachusetts Avenue, to el. -19.0 between Arlington Street and Hadassah Way (its lowest point) to el. -10.0 at Charles Street. Table 2 presents elevation and soil condition data for the subway. A cross-section through the structure between Berkeley and Clarendon Streets is shown in Figure 10.

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

L.B. Manley, Asst. Engineer for the Transit Commission at the time, reported on soil conditions:[8]

> "As is well known, the land reclaimed from the Back Bay consists of sand and gravel filling resting on a bed of silt whose upper surface lies at about grade 0, Boston City base, or grade 100, Boston Transit Commission base. This layer of silt is continuous throughout the length of the subway, and attains a thickness of about 17 ft. at Dartmouth Street, and over 20 ft. in the Fens. Between Exeter Street and Charlesgate East and between Clarendon Street and Charles Street, where it finally disappears, it averages about 8 ft. in thickness. Below the silt between Massachusetts Avenue and Hereford Street, and at Exeter Street, are pockets of peat from 2 to 4 ft. in thickness. Another extensive body of peat occurs between Arlington and Charles Streets, where it attains a great depth.
>
> "Below the silt and peat is a stratum of sand and gravel which also extends throughout the length of the subway excavation except for a length of about 1,600 ft. between Exeter and Clarendon Streets. This sand and gravel carries large quantities of water laden with sulphurated hydrogen, which has been offensive to passersby and injurious to the health of those working in it. This gas, as it leaves the surface of the water, is particularly destructive to metal, and copper floats in several of the temporary pump wells have been corroded through at the surface of the water in a few weeks' time by the action of this gas. It is supposed that this layer of gravel is the same as that which appears in the bed of the Charles River and affords an underground water course which tends to equalize the level of the groundwater in the Back Bay."

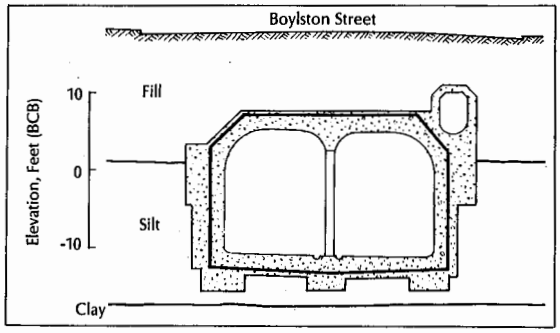


FIGURE 10. A cross section of the Boylston Street Subway at Sta. 58+00 between Berkeley and Clarendon Streets (Ref. 21).

During construction, a temporary drawdown of water levels both in the fill and in the sand-gravel stratum would have occurred. Where the subway route passed opposite to what is now the Prudential site, drawdown in the sand stratum to el. -10.0 is estimated.

Constructed between October 1937 and 1940, the Huntington Avenue subway crosses under Massachusetts Avenue as it enters Back Bay and joins the Boylston Street subway at Exeter Street (see Figure 4). Within this area, the bottom grade of the subway structure

TABLE 2

Boylston Street Subway

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

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

varies from el. -10.0 at Massachusetts Avenue down to el. -19.0 where the structure passes below the railroad tracks (and under the Massachusetts Turnpike Extension). Table 3 presents elevation and soil condition data.

The subway was founded on the outwash stratum that extends from 5 to 12 ft. below the bottom of concrete from Massachusetts Avenue to the Turnpike. North of the Turnpike to Boylston Street, the structure bears on clay and organic soils, without piling. During construction, the outwash stratum was dewatered for the entire length of the subway along Huntington Avenue to grades as low as, or even below, el. -20.0. A very significant drawdown of water level occurred over a wide area, for a period of 2 to 3 years. An observation well at Massachusetts and Commonwealth Avenues, 0.4 miles away, was reported to have dropped from el. 7.0 to el. 0 in 1939.

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

Construction of Storrow Drive in the early 1950s included an underpass and traffic interchange in the Berkeley Street area. This underpass is approximately 1,300 ft. long between

TABLE 3

Huntington Avenue Subway

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

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

portals, with 300-ft. long approach ramps at either end. The road surface descends as low as about el. -4.0, about 15 to 17 ft. below the ground surface.

The underpass was designed to prevent groundwater lowering by the extensive use of waterstops. The structure was designed as a boat with sufficient weight to resist hydrostatic uplift pressure. Invert slabs were up to 2-ft. 8-in. thick. Precipitation and other surface water is collected in catch basins and cross drains that feed into pipes below the slab. These pipes transport water to wet wells near each portal where the water is then pumped into the Charles River.

Soon after completion, leaks were reported in the reinforced concrete walls. In order to collect the infiltrating groundwater and improve the appearance, gutters and false walls were installed. The leaks were evidently never repaired. A significant volume of groundwater is apparently infiltrating into the underpass as recent dry weather pumping volumes have been reported to be about 20,000 gallons per day from each wet well.

The Massachusetts Turnpike Extension, a six-lane limited access highway, crosses the Back Bay. The highway was constructed between 1963 and 1966, and is located just north of the Conrail (formerly Boston and Albany) railroad alignment (see Figure 4). The roadway was depressed 15 to 20 ft. below adjacent city streets and developed areas. The road surface descends from about el. 11.0 at Massachusetts

Avenue down to el. 6.0 at Tremont Street.

The turnpike was designed to prevent a permanent lowering of groundwater levels below about el. 6.5 to 8.5, depending on the location. West of Huntington Avenue, an underdrain system was used to limit uplift pressures on the slab. Through the Prudential Center site, two lines of steel sheetpiling driven 5 ft. into the clay inhibit the flow of groundwater to the turnpike underdrain.

Because the road surface east of Huntington Avenue was lower, underdrains were not used. The turnpike structure was designed for uplift as a boat section, using a thick concrete slab to prevent flotation. A drain was provided along the north wall to prevent groundwater levels from exceeding el. 8.5. Existing drains in the railroad alignment to the south maintain water levels at about el. 7.0.

Southwest Corridor Project. This new transportation structure was constructed between 1981 and 1985. It has two tracks for the relocated Massachusetts Bay Transportation Authority Orange Line subway and three tracks for commuter rail and Amtrak service. Through Back Bay, the alignment followed parts of two original railroad embankments that were constructed across the Receiving Basin in the mid-1830s (see Figure 4 on page 37). From Massachusetts Avenue to Dartmouth Street, the new concrete structure was below ground in a 3,000-ft. long cut-and-cover tunnel that required excavations as deep as 38 ft. East of Dartmouth Street, the structure extended about 10 ft. below former grade. Depths of excavations and other data are summarized in Table 4 on page 46.

Reinforced concrete slurry walls were used for lateral support of the sides for about 2,100 ft. of the tunnel excavation (see Figure 11). The concrete walls were 3-ft. thick and penetrated 8 to 15 ft. into the clay stratum. They were used as the tunnel's permanent outside walls. Although water leakage did occur through some of the vertical joints between wall panels, there was no appreciable lowering of groundwater levels in adjacent areas.

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

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

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

Major Buildings. The first major buildings with deep basements in Back Bay were the Liberty Mutual and New England Life buildings, constructed in the late 1930s. Since that time, other buildings requiring excavation well below the groundwater table have been erected.

Temporary Effects of Building Construction Dewatering

Where excavations have been carried below the water table for building construction in Back Bay, the water table in nearby areas has been lowered, in some cases by a significant amount. Table 5 on page 48 summarizes pertinent information — dates of construction, location, foundation type, elevation of the deepest excavation, dewatering and drawdown — for major construction projects gathered from the available literature, reports

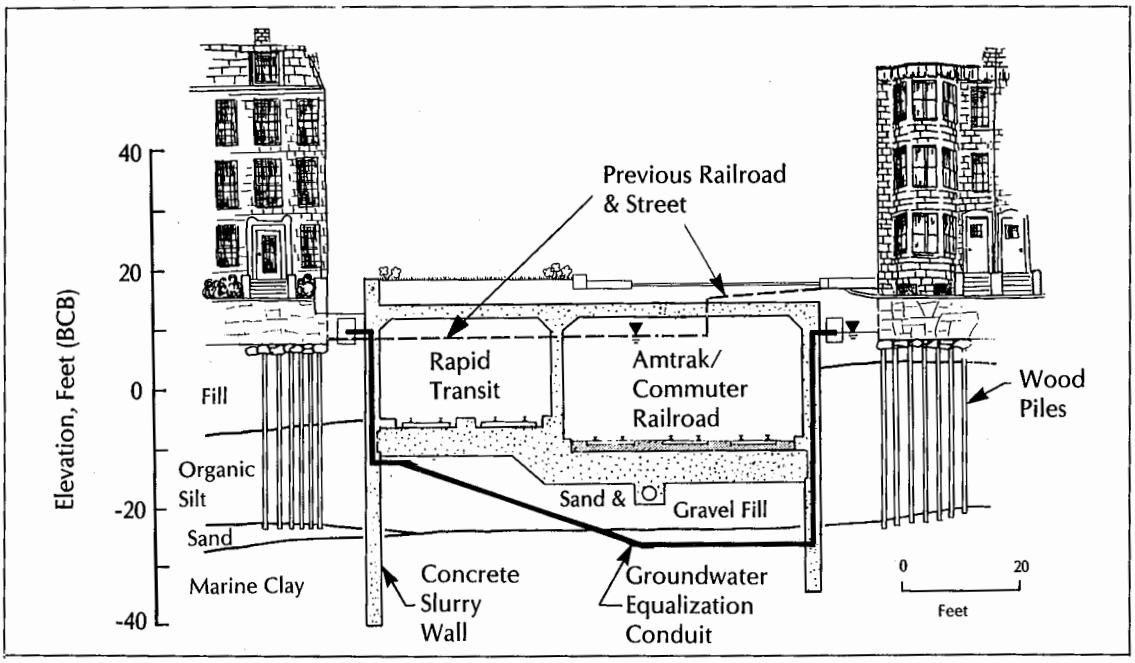


FIGURE 11. A cross section of the Southwest Corridor Project tunnel, taken at Follen Street.

and construction records. The locations of major deep excavations for both buildings and sewer and transportation projects, and the approximate elevations of the bottoms of these excavations are shown in Figure 12 on page 50.

Copley Square Area. The sand outwash is shown in Figure 4 on page 37 to be absent around and for some distance north, east and south of Copley Square, which is generally near the intersection of Dartmouth and Boylston Streets. Therefore, the principal source of water to excavations in the area is by seepage from the fill. Drawdown in the fill is limited to the distance between normal groundwater level and the top of the underlying organic silt, generally less than 7 to 10 ft. Drawdowns of this magnitude were usually confined to areas near the excavation. The volume of water entering excavations has been small and temporary recharging has generally not been practiced.

Excavations for deep foundations have advanced from the early use of unsupported slopes, often combined with deeper laterally supported soldier piles and wood lagging, to the more recent use of steel sheetpiling and concrete diaphragm walls installed in slurry trenches.

In the case of the New England Mutual Life Insurance Co. building, a nearly 40-ft. deep excavation was opened in 1939 over the western two-thirds of the block bounded by Clarendon and Berkeley Streets, and Boylston and Newbury Streets. Steel soldier piles and wood lagging were used to support the sides of the excavation in the organic silt stratum, and the overlying fill was cut back to a stable slope. Surface water and groundwater were collected in troughs cut into the top of the organic silt stratum at the toe of slope behind the sheeting, and sumps were used to dewater the excavation. The building was founded on large spread footings and mats bearing on the stiff crust of the clay at el. -17.0 to -22.0, an early classic example of a "floating" foundation. Adjacent to the excavation, groundwater levels in the fill would have been lowered to the top of the underlying organic silt, about el. 0. Water levels in observation wells located immediately west of the site, at Clarendon Street, dropped 4 to 6 ft. to about el. 2.0. There were no reports of adverse effects to surrounding buildings.

The excavation, in 1947, for the John Hancock Berkeley building was very

TABLE 4

Southwest Corridor Project

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

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

similar to that for the New England Mutual building. The excavation was opened on Berkeley Street between St. James Avenue and Stuart Street. Again, soldier piles and wood lagging were used to support the sides of the excavation in the organic silt, while the overlying fill was cut to a slope of about 1.5 horizontal to 1 vertical. In order to intercept groundwater and surface runoff, an 8-in. pipe was installed around the sides of the excavation in a sand-filled trench located just above the organic silt. The excavation was dewatered using three caisson wells.

During construction, groundwater levels in the fill adjacent to the excavation would have been lowered to the top of the organic silt, as they were during the construction at the New England Mutual building. A 10-ft.

Lateral Earth Support System	Type of Foundation
None, excavation sloped	Belled caissons bearing on clay crust
Steel sheet piling	Rectangular pedestals bearing on clay crust
Concrete diaphragm wall	Slab on compacted gravel borrow replacement fill
Concrete diaphragm wall	Slab on compacted gravel borrow replacement fill
Concrete diaphragm wall	Slab on compacted gravel borrow replacement fill
Steel sheet piling	Precast-prestressed concrete piles to glacial till/bedrock
Soldier piles & lagging to south (Amtrak) side. None to north side.	Precast-prestressed concrete piles to glacial till/bedrock
Soldier piles & lagging to south side. None to north side.	Precast-prestressed concrete piles to glacial till/bedrock
Soldier piles & lagging to south side. None to north side.	Precast-prestressed concrete piles to glacial till/bedrock
Soldier piles & lagging to south side. None to north side.	Precast-prestressed concrete piles to glacial till/bedrock

drawdown, to about el. -2.0 on St. James Avenue, would have occurred. Casagrande reported a 10-ft. drawdown in 1947 across Berkeley Street at the Liberty Mutual Building.[9] To the west of the excavation, groundwater levels in the fill were also lowered by 4.5 and 2.5 ft. at distances of 125 and 300 ft., respectively.

An increase in the rate of settlement of the Liberty Mutual building, located across from the site at Berkeley Street, was attributed to an increase in the effective stress in the clay stratum caused by lowered groundwater levels, and to the effects of disturbance to the structure of the clay from the driving of steel H-piles.[10] The building settled an additional 0.5 in., about half of which was recovered in rebound when the groundwater returned to

TABLE 5

Temporary Effects of Major Back Bay Construction on Groundwater Levels

Construction Project	Location	Years of Dewatering	Site Dewatering: Probable Lowest Elevation	Site Dewatering: Method or Remark
Sewers & Drains				
West Side Interceptor	Charles St. to Beacon to Hereford to Falmouth to Gainsborough St.	1877-1884	Varied; el. -2.0 to el. -7.5	Underdrains & sumps
Low Level Sewers	Throughout Back Bay	1910-1912		Underdrains & sumps
West Side Interceptor Relocation	At Christian Science Church Center	1968-1969	el. -7.5	Wellpoints in sand
Subways				
Boylston Street Subway	Boylston St. from Arlington St. to beyond Mass. Ave.	1912-1914	el. -19.0	Deepest between Arlington St. & Copley Square
Huntington Avenue Subway	Beneath Exeter St. & Huntington Ave. to beyond Mass. Ave.	1937-1940	el. -20.0	Lowest under Mass. Turnpike & railroad
Huntington Avenue Subway Prudential Entrance	Huntington Ave. near West Newton St.	1969-1970	el. -15.0	
Southwest Corridor Project	Mass. Ave. to Harcourt St.	1981-1985	el. -15.0 to el. -28.0	None required
	Harcourt St. to Berkeley St.	1982-1985	el. -28.0 el. -4.0	Sumps in excavation
Buildings				
Christian Science Publ. House	Norway St.	1931-1934	el. -3.0 to el. -6.0	Wellpoints in Sand
New England Mutual Life Ins. Co. Bldg.	Clarendon St. between Newbury & Berkeley Sts.	1939-1940	el. -21.0	Sumps in excavation
John Hancock Berkeley Bldg.	Berkeley St. between St. James & Stuart Sts.	1946-1947	el. -25.0	Sumps in excavation
Boston Herald Traveler Bldg.	Harrison Ave. between Herald & Traveler Sts.	1957	Deep in sand above bedrock	
Christian Science Publ. House Underground Equip. Rm.	Norway St. adjacent to Mother Church	1958	el. -3.0	Wellpoints in Sand
Prudential Center Tower	Center of Prudential Center complex	1959-1960	el. -12.0	Wellpoints in Sand
Sheraton Hotel at Prudential Center	Dalton & Belvidere Sts.	1962-1963	el. -4.0	Wellpoints in Sand
180 Beacon St. Apart. Bldg.	Beacon & Clarendon Sts.	1964-1966	el. -14.0	Unknown
Christian Science Admin. Bldg.	Huntington Ave. between Belvidere & Cumberland Sts.	1967-1968	el. -5.0	Open pumping in excavation
Christian Science Colonnade Bldg.	Clearway & Belvidere Sts.	1968-1969	el. 0.0	Shallow, probably by sumps
John Hancock Tower	Clarendon, between St. James & Stuart Sts.	1968-1974	el. -28.0	Sumps within site
Christian Science Church New Portico	West side of Mother Church	1973	el. -4.0	Wellpoints in Sand
Symphony Plaza Apartments	Mass. Ave. at St. Botolph St.	1977	el. +4.0 to -3.0	Sumps in pile cap pits
Greenhouse Apartments	Huntington Ave. at W. Newton St.	1981	el. -2.0	Sumps in pile cap & elevator pits
Copley Place	Huntington Ave. between Harcourt & Dartmouth Sts.	1981-1982	el. -4.0	Sumps in pile cap excavations
One Exeter Place	Boylston & Exeter Sts.	1982-1983	el. 0.0	Sumps in excavation

Lateral Earth Support System		Effect of Pumping on Groundwater Levels		
Type	Lowest Stratum Penetrated	Distance from Excavation	Drawdown	Other Remarks
Probably wood sheeting	Organic Silt & Sand	No records available		Signficant drawdown in adjacent areas probably occurred. See Figure 4.
		No records available		Significant drawdown in adjacent areas probably occurred.
Steel sheet piles	Clay	100 ft. 700 ft. 100 ft.	8 to 10 ft. in Sand none in Sand 3 ft. in Fill	Recharge around Christian Science Church eliminated drawdown in Fill.
Steel sheet pile	Clay	No reports available		Siphon pipes installed to allow groundwater movement across structure. See Table 2.
Steel sheet piles	Clay	200 ft. 800 ft. 1100 ft. 0.4 mi.	13 ft. in Sand 12 ft. (after 2 yrs.) 8 ft. in Sand 7 ft. in Sand	Dewatered for 3 yrs. WPA data indicated large area affected (Mass. Ave. to Dartmouth St.). See Table 3.
Steel sheet piles		300 ft. 400 ft. 800 ft. 1000 ft. 1200 ft. 1400 ft.	18 ft. in Sand 4 ft. in Sand 6 ft. in Sand 8 ft. in Sand 6 ft. in Sand 4 ft. in Sand	Dewatering for 5 months. No drawdown observed in Fill.
Concrete diaphragm walls	Top of Clay	None observed that was attributable to corridor tunnel construction.		Excavation open 1-2 yrs. Observ. wells at each street. See Table 4.
Sheet piles, soldier beams & lagging	Organic Silt			Rises to el. -4.0 east of Dartmouth St.
Unknown	Sand	300 ft. 300 ft. 1000 ft.	8 ft. in Sand 2.5 ft. in Fill 4-5 ft. (in Sand?)	
Soldier piles & lagging	Clay	Nearby	4-6 ft. in Fill	No Sand at site.
Soldier piles & lagging		90 ft. 140 ft.	10 ft. 7 ft. in Fill	No Sand at site.
	Unknown	1 mi.	30 ft. in deep sand	Dewatering for caissons to rock.
Steel sheet piles	Sand, halfway through	At excavation 500 ft. 1200 ft.	11 ft. in Sand 8 ft. in Sand 4 ft. in Sand	Little drawdown in Fill.
Steel sheet piles	Clay	Nearby	16 ft. in Sand 1-2 ft. in Fill	Recharged Fill & Sand outside sheeting.
Steel sheet piles	Clay	400 ft.	Initially, 3.5 ft. in Sand	Recharge system correction eliminated drawdown.
Concrete slurry walls	Unknown	Nearby	12-15 ft. in Sand	Slurry wall leaked. Recharged Sand outwash unsuccessful.
	Unknown	200 ft. 500 ft.	"Some" in Fill 0 ft. in Fill	
	Unknown		"Minor" in Fill	No data available on off-site drawdown.
Steel sheet Piles	Clay	Near	Negligible in Fill	No Sand at site.
Partially sheeted	Sand	40 ft. 90 ft. 230 ft.	8 ft. in Sand 4.5 ft. in Sand 5.5 ft. in Sand	No drawdown observed in Fill.
	Unknown	Nearby	3 ft. in Fill	
Soldier pile & lagging	Organic Silt	40 ft.	3 ft. in Fill	
Soldier pile & lagging	Organic Silt	Near	Minor in Fill	Only pile cap excavations went below groundwater.
Steel sheet piles	Clay	Nearby	Negligible in Fill	

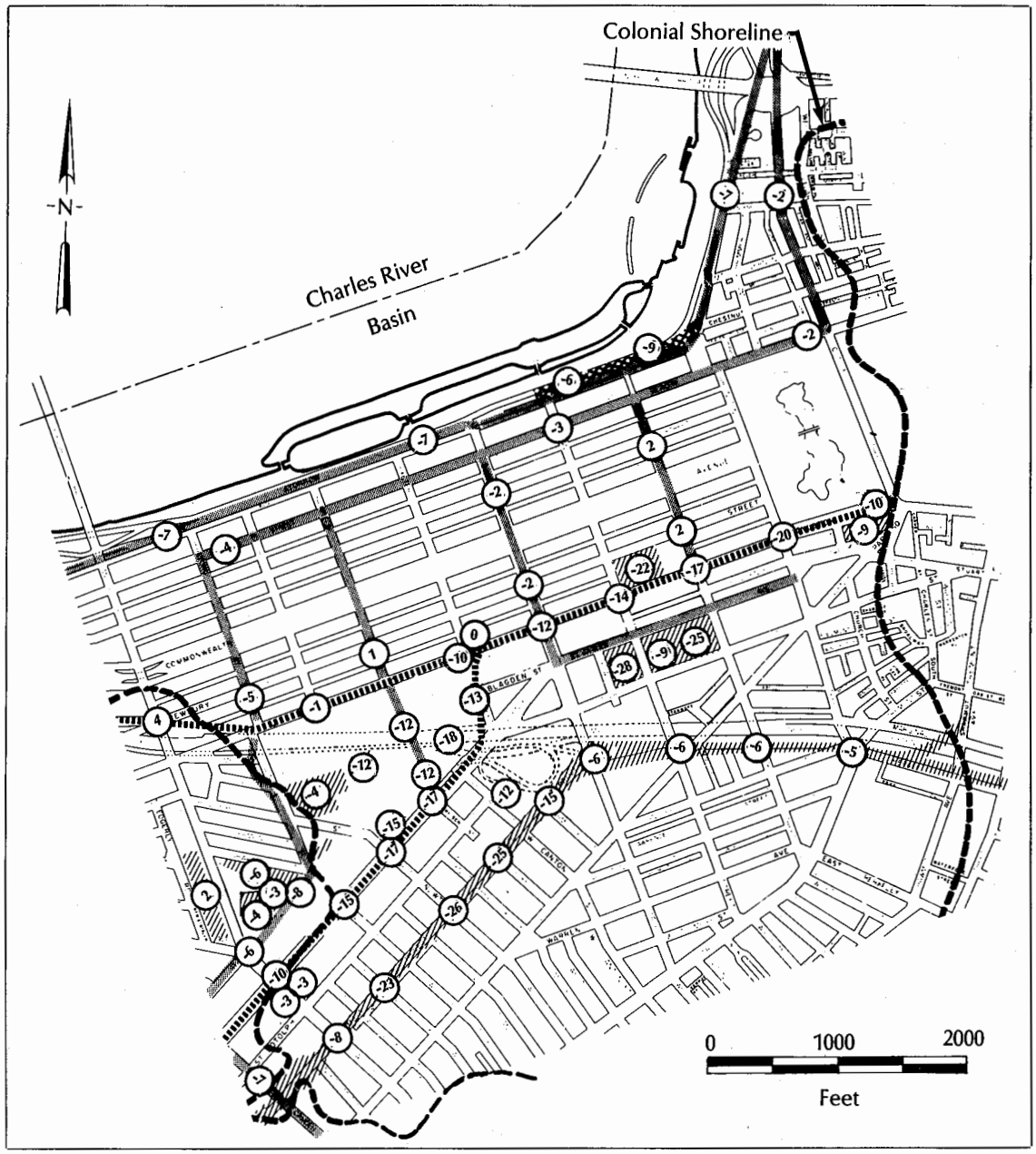


FIGURE 12. The locations and elevations of major deep excavations in Back Bay.

pre-construction levels.

Construction of the John Hancock Tower, begun in late 1968, required excavation to el. -28.0 (45 ft. below ground surface).[11] Interlocking steel sheetpiling was driven approximately 5 ft. into the clay to form a cofferdam around the site. The sheeting extended to ground surface, unlike excavations at the New England Mutual and the Hancock Berkeley buildings where the fill was sloped down to the laterally-supported organic silt. Prior to construction, during October and November 1967, water levels at the site varied between el. 4.5 and 6.0, with an average of el. 5.0.

Contrary to experience at the Prudential Center, there was no significant drawdown of the water table beneath the streets surrounding the site. Water levels measured in the fill

during construction were generally from el. 4.0 to 5.0. Immediately behind the steel sheeting, a local drawdown exceeding 10 ft. was measured in a piezometer installed in the organic silt. Very little pumping was required inside the sheeting and no recharging was performed. Drawdown at the John Hancock Tower was insignificant because steel sheet piling was used and the sand outwash stratum was absent.

Foundation construction for Copley Place began in 1981 on a 10-acre site bounded by Huntington Avenue, the Southwest Corridor, Harcourt and Dartmouth Streets. The lowest floor level was established at el. 6.6, somewhat above the pre-construction groundwater table. Dewatering was required only for the construction of pile caps and a deep water main. Excavation and dewatering for one large pile cap below the Westin Hotel was carried to approximately el. -5.0, using steel sheeting for lateral support. Elsewhere, the sides of the excavation were either unsupported slopes or supported with soldier piles and wood lagging.

The impact of dewatering on groundwater levels adjacent to the site was minor. An observation well on Blagden Street adjacent to the Boston Public Library dropped temporarily about 3 ft. However, there was no observable drawdown at Trinity Church.

A quarter-mile north of Copley Square, at the corner of Beacon and Clarendon Streets, significant construction was required for a 17-story apartment building constructed in 1964-1966 at 180 Beacon Street. Here, soil conditions were much like those in Copley Square. Concrete diaphragm walls installed in slurry trenches were used both for lateral support of the sides of excavation and as permanent basement walls, the first such use in Boston. The 2-ft. thick reinforced concrete walls were internally braced and surrounded the site. These walls penetrated about seven feet into the clay stratum, which is overlain at this location by 5 ft. of sand outwash. The site was excavated down to about el. -20.0 for the 3-½ basement levels required (12 to 14 ft. above the outwash stratum).

Leakage through the concrete wall was apparently the cause of a 12 to 15-ft. drawdown in observation wells installed in the outwash on adjacent property. Water from city mains was pumped into the outwash through five 2-inch diameter recharge wells, but with only moderate success in raising the piezometric head. The extent to which the perched water level in the fill was affected is not known. Some minor settlement of an adjacent wood pile-supported 10-story apartment building was attributed to the construction, but the cause was never clearly established.

The Christian Science Church Center is southwest of Copley Square. In this area, the pervious sand outwash stratum is fully developed to a thickness of 12 to 20 ft. Most of the structures built in the Christian Science complex have been founded, in one way or another, on this outwash. The Mother Church was founded on untreated wood piles. Dewatering of the confined outwash aquifer had been required during construction of several buildings constructed in the last 55 years in this complex.

In 1932, excavation for the 100 by 630 ft. Christian Science Publishing House building on the former Norway Street was carried as low as el. -6.0 for spread footing construction on the sand outwash. Dewatering by wellpoints in the outwash dropped the piezometric head an estimated 10 to 15 ft. for approximately eight months. The lateral earth support system used for this excavation is not known.

Water levels in the confined outwash aquifer responded quickly to the dewatering over a large area. The water level in an observation well located 1,200 ft. southwest of the site dropped 4 to 5 ft. in two weeks, while at the Mother Church, the water level was lowered 8 ft. The effects of construction dewatering in the fill were much less, with the water table dropping only 2.5 ft. at the Mother Church.

Drawdown in 1958 for the excavation of the Christian Science Publishing House Underground Equipment Room adjacent to the Mother Church was similar to that experienced in 1932 during Publishing House construction. Excavation extended to el -3.0 in the sand outwash. Lateral earth support was

provided by steel sheet piling that penetrated part way through the sand. Wellpoints were used inside the sheeting to dewater the outwash to approximately el. -4.0. Water levels in observation wells in the outwash 500 and 1,200 ft. from the site were lowered to el. -1.0 and 3.0, respectively.

As part of a major construction program at the Christian Science Church between 1969 and 1972, an approximately 1,000-ft. long section of the West Side Interceptor was relocated into a gallery. Construction was carried out between two rows of steel sheet piling, 16 ft. apart, and driven 5 ft. into the clay stratum. The bottom of the foundation for the gallery was el. -7.5 in the sand outwash. Dewatering was accomplished by wellpoints installed inside the cofferdam. Drawdown in the sand stratum at the nearby Mother Church was initially to el. -2.0. Six large-diameter recharge wells installed around the Mother Church raised piezometric levels to about el. 3.0. Later, as recharge became ineffective, outwash water levels fell back to el. -2.0 and -3.0. In the fill stratum, the initial drawdown to el. 3.0 was successfully recharged to el. 6.0.

For the 1973 construction of a new portico for the Mother Church, the last major project at the Christian Science complex, foundations at the front of the Mother Church were underpinned with six concrete piers bearing on the outwash at el. -3.5. Dewatering by wellpoints lowered water levels in the outwash by 8 and 5.5 ft. at distances of 40 and 230 ft. from the excavation, respectively. There was no observable drawdown in the fill.

The Prudential Center is immediately west of Copley Square. Construction began in 1959 with a 52-story tower. The entire development, the largest in Boston at the time, was enclosed within a wall of interlocking steel sheeting that was reported to have been driven 5 ft. into the clay stratum to form a relatively impermeable cofferdam. Internally, the area was divided by sheet pile walls into several cells.

A parking garage was constructed beneath most of the Prudential Center, with the lowest floor level at el. 3.0. A portion of the slab was supported on compacted sand and gravel fill that was placed after the organic soils were excavated. This excavation and backfill operation, and other excavations requiring dewatering, were accomplished with wellpoints.

Drawdown in the outwash sand just outside the sheet piles was reported to have been to as low as el. -12.0. Construction specifications required recharging outside the sheeting to maintain groundwater levels at or above el. 5.0. There were considerable problems with recharging the sand outwash, and it was only moderately successful. However, there were no significant problems in maintaining water levels outside the sheeting in the fill.

In 1969 and 1970, an area at the edge of the Prudential Center was dewatered for the construction of a new entrance to the Huntington Avenue subway. Excavation and dewatering were carried out to el. -15.0 in the sand outwash. Dewatering lowered water levels in wells at the Mother Church and Massachusetts Avenue to el. -7.0 and 0.0 (1,000 and 1,200 ft. away, respectively). Drawdown in the sand outwash was reported to have caused the Prudential Center garage to settle 0.5 in. Here again, most of the subsidence was recovered after dewatering when water levels returned to pre-construction levels.[9,12]

Effects from Outside Back Bay. In 1957, the Boston Herald Traveler Corporation started construction of a new building at 300 Harrison Avenue, well outside of the fill area to the east of Copley Square. Dewatering of the glacial till/outwash strata occurring *below* the clay was required for construction of deep caissons. Because these materials are relatively pervious, piezometric levels in the till and outwash were lowered significantly in Back Bay. Water levels in deep observation wells at the Prudential Center, approximately one mile away, dropped as much as 30 ft. within a month of the start of pumping.[9] This drawdown over a period of five months caused about 0.1 and 0.3 in. of settlement at the New England Mutual Building and the Liberty Mutual Building, respectively, located about 0.8 and 0.6 miles from the Herald Traveler Building.[10] Most of the subsidence was recovered by rebound after dewatering.

Overall, for sites where the sand outwash stratum was absent, for example in the Copley Square area, deep excavations have been successfully accomplished within steel sheet pile cofferdams without significant groundwater drawdown in the fill and without having to install recharge systems. Where the outwash occurred to the west, even the use of steel sheeting had not prevented drawdown in the pervious sand stratum because of leakage through untensioned interlocks.

Drawdown in the sand outwash stratum generally extended 5 to 10 times further from a deep excavation than groundwater drawdown in the fill. Water levels in the outwash were lowered significantly at distances of 1,000 ft. or more from an excavation. In the fill, however, drawdown greater than one foot did not usually occur at distances beyond 400 ft. Drawdown in the fill was limited by the depth to the organic silt stratum. Groundwater recharge systems have been successfully used to limit lowered water levels in the fill. Similar systems have not been effective in the outwash stratum.

Historical Groundwater Levels in Back Bay

Groundwater levels in Back Bay are influenced by the natural process of precipitation and infiltration, and by water levels in adjacent bodies of water. If there were no man-made structures, the water table would be relatively uniform across Back Bay and would vary little with time, being affected only by the amount of precipitation and local infiltration.

Construction over the past 100 years — sewers and drains, dams, transportation corridors and building foundations described above — have diverted or withdrawn groundwater, have impeded its flow and, in other respects, have influenced the water table. Groundwater levels are non-uniform, complex and have varied substantially in localized areas over time. Therefore, the interpretation of groundwater data can be confusing, frustrating and misleading.

Concern for groundwater levels in Back Bay has prompted sporadic action during the past 100 years. Area-wide studies were made before and after construction of the Boston Main Drainage System in the late 1800s, during the 1890s for the Charles River Dam, in the late 1930s under a WPA program, by the USGS in 1967 and 1968, and in 1985 for the BRA.[13] In the past 25 years, numerous studies have been undertaken in local areas for building construction, most recently in 1985.[18] However, there has been no long-term study.

1880s Study. Stearns reported that groundwater levels in wells installed in 1878 before construction of the main drainage system were practically the same in 1885, one year after construction, with water levels "nearly level at Grade 7.7 over the whole district" (p. 26).[3] The data indicated levels between el. 6.7 and 8.5. Engineers of that time realized the importance of maintaining groundwater levels and were concerned about the effects of the new main drainage works on the water table.

1894 Charles River Dam Study. Water levels were measured in wells installed for an 1890s study of the proposed Charles River Dam as reported by Stearns.[3] Generally, groundwater levels were similar to those measured between 1878 and 1885. Stearns blamed leaky sewers for some levels below el. 5.0, but considered these instances to be local and isolated. He recommended that el. 8.0 be established as the water level for the new Charles River Basin. In discussions to Worcester's 1914 paper, Gow and Stearns cited leaky sewers as a cause of local groundwater depressions.[8]

1930s Copley Square Study. In 1929, public officials and residents noticed several alarming cracks in the Boston Public Library Building, and settlement of the stone platform in front of the library facing Copley Square on Dartmouth Street. Investigations by the Building Department and consulting engineers found that the tops of many wood piles supporting the building were completely rotted away or badly decayed.[14,15] Piles were originally cut off at approximately el. 5.0 and groundwater was found to average el. 4.0 at the time of underpinning.

Rotted piles below approximately 40 percent of the building area were cut off to sound wood and were posted with 6-in. steel H-sections bearing on steel plates and wedged against the underside of the stone foundation. The cost of this underpinning in 1929-30 was

reported to be "nearly $200,000."

The discovery of rotted wood piles under the Library sparked renewed interest in groundwater levels, especially among the Trustees of the nearby Trinity Church that was constructed in 1876 on 4,500 wood piles. Numerous observation wells were installed in the Copley Square area, showing water levels as low as el. 2.0.

When contours of equal water level were analyzed (see Figure 13), the loss of groundwater was traced to leakage into a 30-in. diameter sewer on St. James Avenue (see Figure 9). Construction of a partial dam in the sewer on Dartmouth Street, where it joins the Boylston Street sewer in front of the Public Library, caused observation wells to rise immediately, proving without a doubt the source of the lost groundwater.

Fortunately, Trinity Church was spared. Excavations to examine the condition of wood piles, originally cut off from el. 5.0 to 5.5, showed no significant deterioration. The structure had settled nearly one foot in 50 years and pile butts were now lower. This case history was documented by Robert Treat Paine in 1935.[16] In addition, Snow traced the loss of groundwater in the area.[14]

1936-1940 WPA Surveys. The city of Boston measured groundwater levels throughout the Boston Peninsula between 1936 and 1940. The project was funded by the Works Progress Administration under projects No. 5325 and No. 188868. The impetus for this study was the growing concern about groundwater levels in the city during the 1920s and 1930s, heightened by the discovery of the rotted wood piles at the Boston Public Library.

Observation wells installed for the WPA project and wells previously installed by the Boston Sewer Department were monitored. Throughout the Boston Peninsula, a total of approximately 700 observation wells were used in the WPA survey. Approximately 300 of these wells were located in Back Bay. A report prepared for the Boston Redevelopment Authority (BRA) contains tables and plans that describe the location of each well, and the highest and lowest water levels recorded during the four-year monitoring period.[13] Unfortunately, complete records of all water levels recorded during the program are not available, since they were destroyed in a fire at Boston City Hall. Figure 14, on page 56, prepared from contour plans by Cotton and Delaney, shows areas in Back Bay having water levels below el. 5.0 during that 4-year period.[4]

Most wells in Back Bay experienced a water level below el. 5.0, and, in seven wells, the *highest* water level measured was also below el 5.0. Local drawdowns from leaking sewers are the most probable cause for the low water levels in the seven wells. Precipitation during the period was about average for Boston, with yearly deviations up to 6 in.

Significant construction projects during 1936-1940 included the Huntington Avenue subway that dewatered the sand outwash to el. -20.0 or below; the Liberty Mutual building, with excavation below el. 0; and the New England Mutual building where excavation extended to el. -21.0. These projects cannot, however, account for the broad extent of low-water levels in Back Bay. Leaking sewers and pumping from sumps in basements of buildings undoubtedly were major contributors.

Dewatering of the pervious outwash stratum for subway construction was probably responsible for the lowered groundwater levels north of the Southwest Corridor alignment from Massachusetts Avenue to Clarendon Street. Groundwater drawdown immediately adjacent to the Huntington Avenue excavation is not known because data are not available for wells there during the construction period.[4]

Throughout much of this area, the outwash stratum is particularly well developed and is separated from the fill by a relatively thin layer (as little as 3 ft.) of organic silt and/or peat. However, in many locations, where trenches and holes have been excavated, the outwash and fill strata are connected and lowered water levels in the outwash can directly affect water level in the fill. Some of the WPA observation wells may have been installed into the outwash stratum. Water levels observed in some wells may, therefore, have been lower than groundwater levels in the fill.

In the Copley Square area, south of Boylston Street between Dartmouth and Berkeley Streets, the low groundwater levels

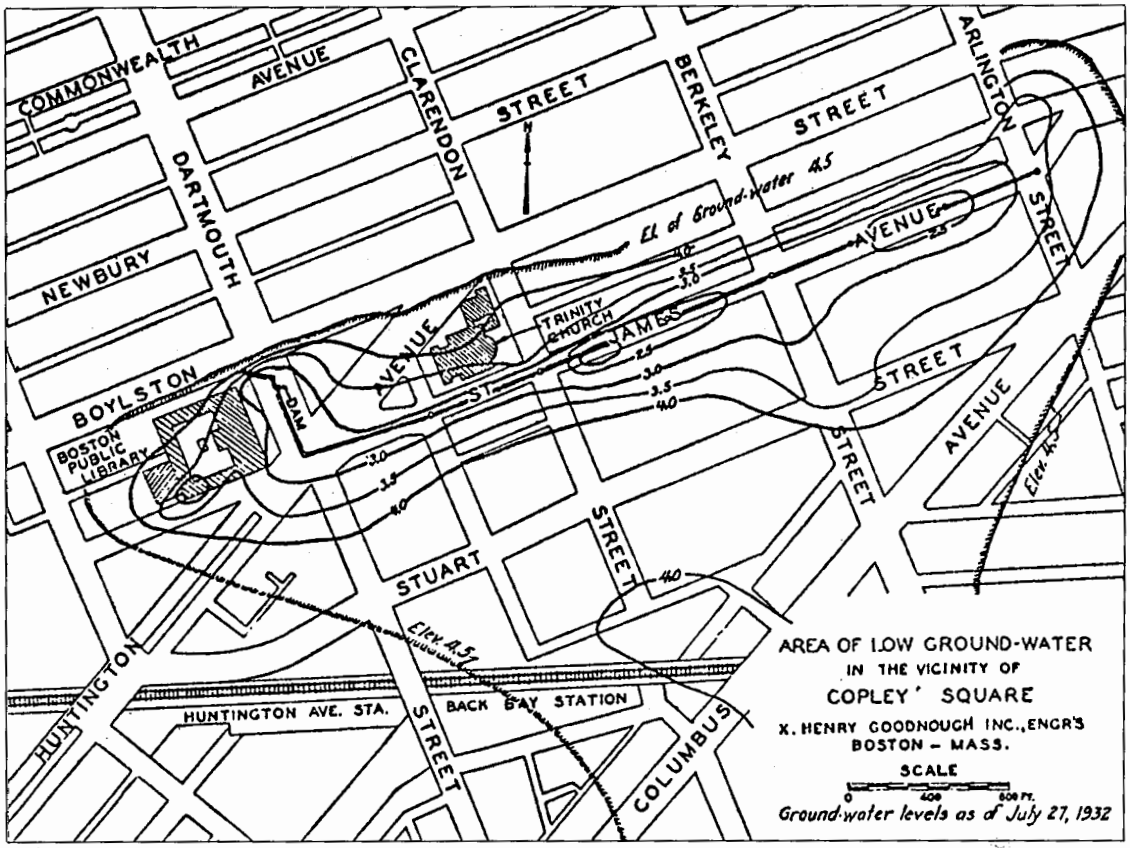


FIGURE 13. Water level contours along St. James Avenue in 1932.[16]

may have been caused by leakage into sewers and drains and pumping from sumps in building basements. Because the outwash stratum generally does not extend into this area and the Boylston Street subway structure forms a barrier to groundwater seepage from across the street, low groundwater levels in this area were probably not related to construction dewatering and drawdown.

The St. James Avenue sewer has a history of causing local groundwater lowering. Groundwater levels have also been lowered by drainage in a crawl space along the easterly side of the John Hancock Clarendon building that reduces hydrostatic pressures on basement floors and walls. Until 1984, the water level had been held at el. -0.5. It has been raised somewhat with recent renovations to the structure. Sump pumping has also been performed at the YWCA building at Stuart and Clarendon Streets.

In other areas, low groundwater levels were probably due to leakage into sewers. The West Side Interceptor beneath Beacon and Charles Streets may have been responsible for low groundwater levels in that area. Low groundwater levels along Tremont Street in the South End were probably caused by leakage into major sewers that join in that area. Some water levels there were as low as el. 0 to -3.0. In this area, there were also several groundwater mounds, probably due to water main leaks. These local recharges intermittently interrupt the drawdown pattern toward Tremont Street.

1967-1968 USGS Measurements. On two occasions, in September 1967 and March 1968, the United States Geological Survey (USGS) measured groundwater levels throughout the Boston Peninsula. This study was made in response to a request by the Massachusetts Department of Public Works which was concerned about the potential adverse effects of construction for the then proposed Inner

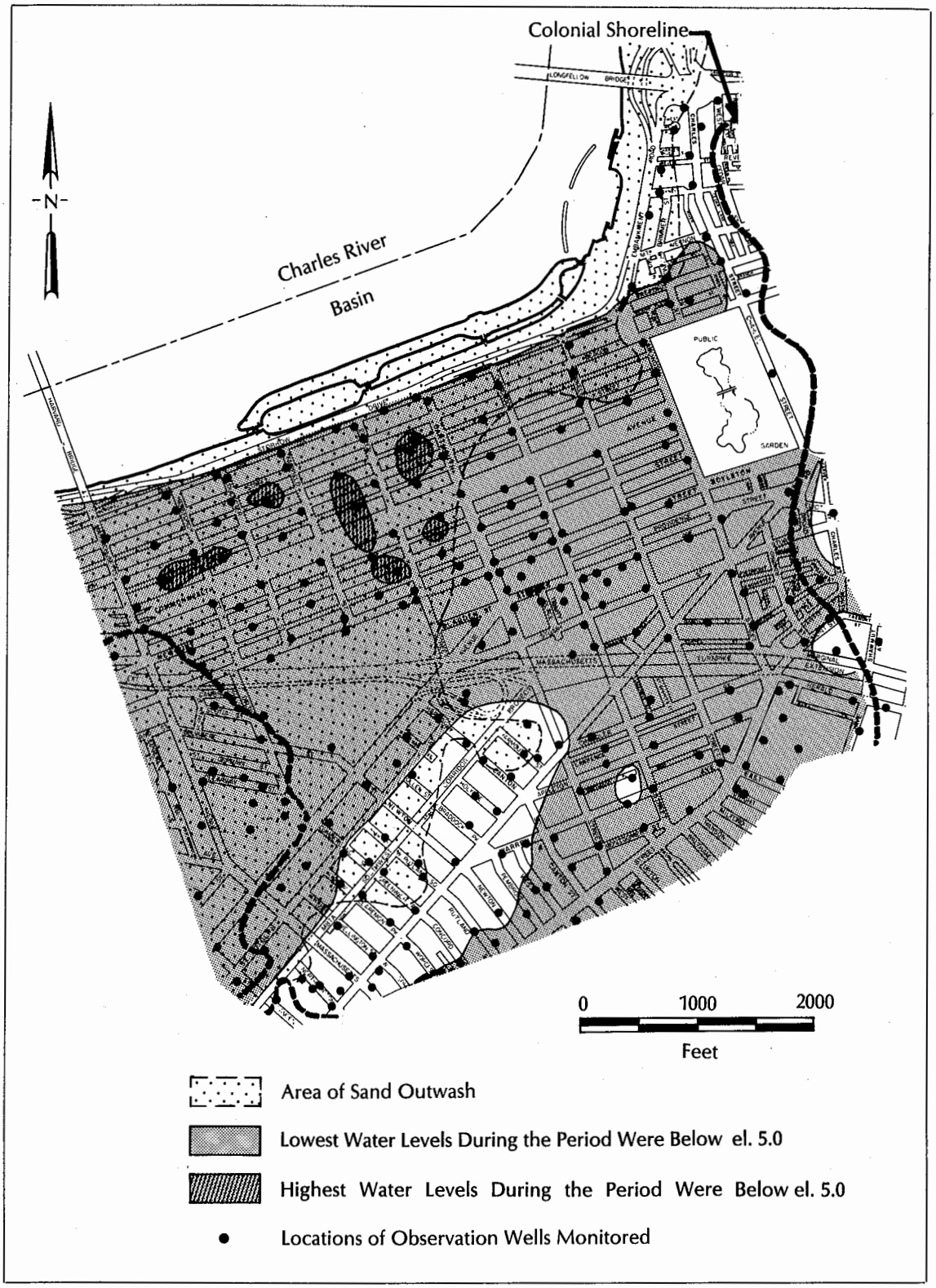


FIGURE 14. Areas of Back Bay having groundwater levels below el. 5.0 during 1936-1940.

Belt expressway on groundwater levels. The USGS used observation wells extant from the WPA survey completed in 1940. Less than half of the original wells were found to be usable. Results of the USGS study were published in 1975 as *Hydrogeologic Investigation Atlas HA-513.*[4] Figure 15 has been prepared from the groundwater contours presented therein.

Areas in Back Bay where the September 20-21, 1967, water levels were below el. 5.0 are shown in Figure 15. Areas where both readings, the second on March 20-22, 1968, were below el. 5.0 are also indicated. In addition to the areas shown, low levels were observed in wells around the Christian Science Center; however, those data may not have been available to the USGS.

It would appear, by comparison of Figure 15 with Figure 14, that water levels throughout Back Bay were higher in the 1960s than in the 1930s. Note, however, that Figure 15 was based on two isolated readings while the 1930s data represent extreme lows from numerous readings over a four-year period. Furthermore, the 1960s readings were taken during a wet period; precipitation in 1967 was 6 in. above normal and 5 in. of rain fell on March 17-18, 1968. On the other hand, looking at the high water level readings, two of the seven wells that were never above el. 5.0 in the 1930s, were above that level in the 1960s. No significant construction dewatering during the 1967-68 period has been reported. Groundwater levels were again below el. 5.0 in the John Hancock area and along Tremont Street in the South End.

Construction within the study area between 1940 and 1967, which includes the Prudential Center, does not appear to have permanently lowered groundwater levels below el. 5.0 by 1967.

1970-1985 Groundwater Levels. The BRA report summarized available groundwater data from numerous building projects.[13] Water level observations were generally made at the building sites and at immediately adjacent areas before, during and shortly after construction. Table 5 lists major projects constructed both before and during the period.

In addition, data were collected from other sources where monitoring is ongoing, for example, at the Christian Science Church, Prudential Center, Boston Public Library, Trinity Church, Massachusetts Turnpike Extension and Church of the Advent. Primarily, data were available for the area along the central spine across Back Bay. Essentially, no recent groundwater data are available for the Back Bay Historic District and other important areas having buildings founded on wood piles.

Figure 16 shows the location of most of the observation wells monitored at some point during the 15-year period from 1970 to 1985 and the area where the lowest water level observed was below el. 5.0. The monitoring period often lasted less than a year, since the purpose of the monitoring was to monitor the effects of construction and dewatering that caused the temporary lowering of water levels in areas adjacent to sites. Some data in Figure 16 were affected by construction dewatering while the two readings in 1967-68 (see Figure 15) were not affected.

Groundwater levels substantially below el. 5.0 in the area of the Christian Science Center and in the Park Square area south of the Public Garden were due to construction-related dewatering. Observed low groundwater levels around Hadassah Way were due to sump pumping from a basement in that area. Groundwater levels of approximately el. 3.0 were observed within the Prudential Center, probably caused by leakage into the underground parking garage. The effect of these low levels on groundwater in adjacent areas is mitigated by a wall of steel sheetpiling that encloses the Prudential Center site. (Note that the USGS observations shown in Figure 15 did not include the Prudential Center.)

Other areas of low groundwater include the area bounded by the Prudential Center, Dartmouth Street, Boylston Street, and the Massachusetts Turnpike, and the block occupied by the John Hancock Clarendon and Berkeley buildings. The drain between the two older Hancock buildings continues to cause lowered groundwater levels in that area. During subsurface investigations for the Copley Place project, low groundwater levels between the Boston Public Library and the Massachusetts Turnpike were concluded to have largely been due to leakage into the

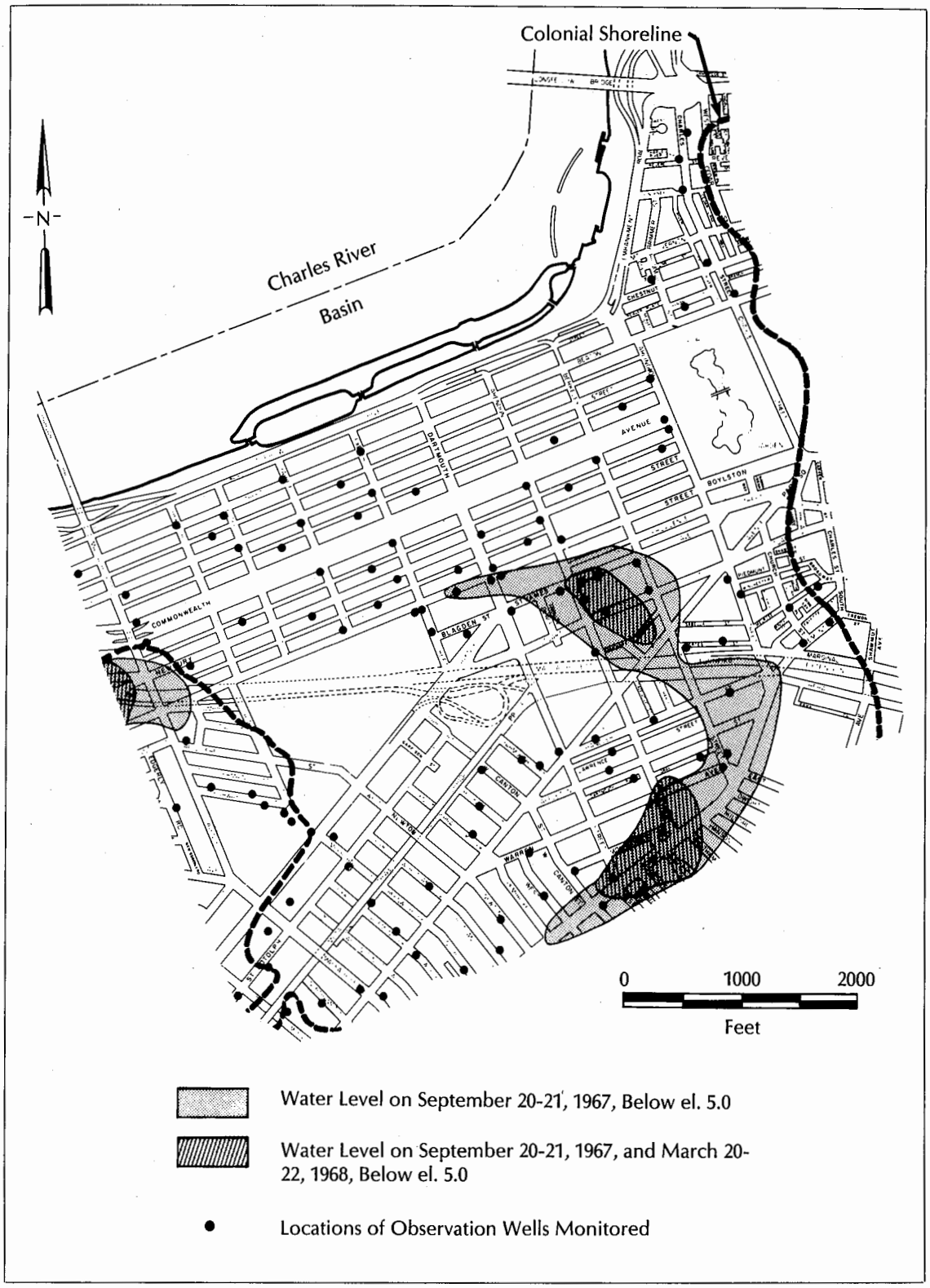


FIGURE 15. Areas of Back Bay having groundwater levels below el. 5.0 during 1967-1968.

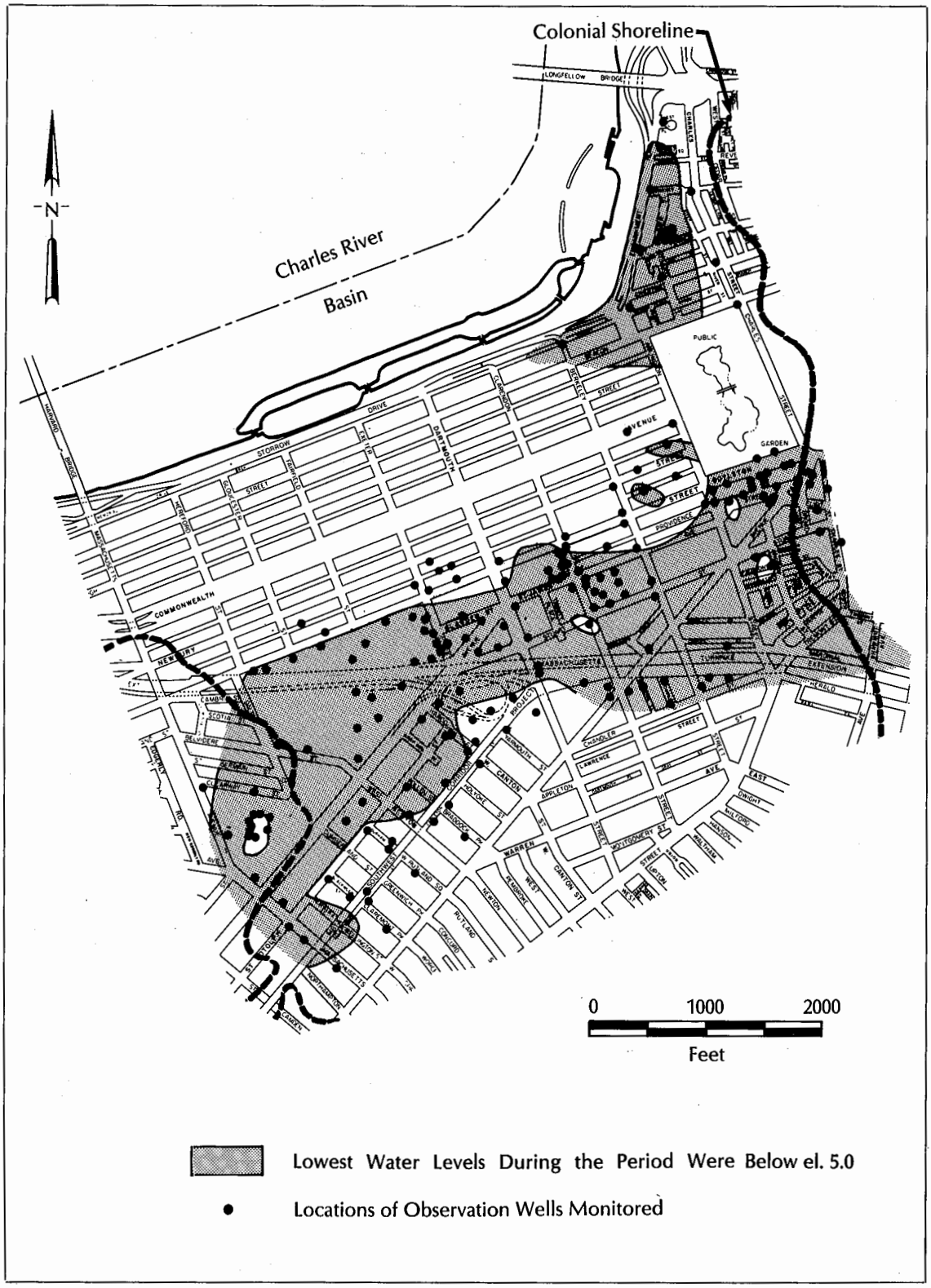


FIGURE 16. Areas of Back Bay having groundwater levels below el. 5.0 during 1970-1985.

St. James Avenue sewer. The Prudential Center, the subway tunnel beneath Exeter Street and the Conrail alignment may also be lowering groundwater levels in this area.

East of the Back Bay railroad station, groundwater levels 1 to 2 ft. below el. 5.0 have been observed on both sides of the right-of-way occupied by the Massachusetts Turnpike and the Southwest Corridor Project. Groundwater levels in this area were observed to be below el. 5.0 for several years before Southwest Corridor construction began and therefore do not reflect construction-related groundwater lowering. Drains in the former railroad right-of-way were probably responsible for the lowered water levels. Massachusetts Turnpike drains were probably not the cause since they are located above the observed low water levels.

Along Tremont Street in the South End, where groundwater levels had been below el. 5.0 in the two previous monitoring periods, data were available for only one observation well. Low water levels in this well, near the intersection of Berkeley and Tremont Streets, were below el. 5.0. These findings could indicate that lowered groundwater levels along Tremont Street still exist.

Recent Lower Beacon Hill Study. In early 1984, attention was focused once again on foundation problems caused by lowered groundwater levels, on this occasion in the lower Beacon Hill area from Charles Street to Embankment Road, bounded to the south by Beacon Street. Residents along the waterside of Brimmer Street, between Pinckney and Mt. Vernon Streets, became alarmed when cracks developed in interior and exterior walls and when other evidence of differential settlement appeared. Test pits were excavated to enable visual examination of the wood piles. In most cases, the wood in the top 1 to 3 ft. of the piles was severely decayed. Groundwater levels were found to be several feet below the pile tops and as much as 6 ft. below the water level in the Charles River Basin.

The principal cause of lowered groundwater has been determined to be leakage through cracks and joints in combined sewer overflows, where they join the Boston Marginal Conduit at the foot of Pinckney and Mt. Vernon Streets. The Metropolitan District Commission (MDC), Massachusetts Water Resources Authority (MWRA) and Boston Water and Sewer Commission have investigated the problem and are taking steps to correct the loss of groundwater.

Occurrences of Rotted Wood Piles

Except for well-publicized cases, records of wood pile deterioration are buried in Boston Building Department files, in the files of building owners, their architects, engineers and contractors, or they do not exist. Owners are understandingly reluctant to talk about the problem.

Six thousand applications for building permits filed with the building department between 1979 and 1984, and representative samples of permit data from 1967-1972 and 1976-1979 were examined in the preparation of the 1985 BRA report.[13] Only two of the permits issued were for repairs to wood piles, suggesting that the problem in recent years (to 1984) has been minor or not reported.

With the exception of the lower Beacon Hill area, other cases throughout Back Bay appear to have been isolated and infrequent. In addition to the Boston Public Library problem in 1929-30, J.R. Worcester identified two occurrences of rotted wood piles:[15]

> "Such extensive repair work has been necessary in other localities in the Back Bay; *e.g.*, the Fire Insurance Protective Headquarters at 4 Appleton Street (South End) in July 1921 had to be underpinned where piles cut as low as elev. 3.96 were rotted off above ground water level found at the time at elev. 3.30; again at 12 Hereford Street, corner of Beacon Street, in June 1933, piles cut at elev. 8.13 were rotted to within 3" of ground water level found to be at elev. 6.50."

Additionally, four buildings between Boylston and Beacon Streets are known to have had deteriorated wood piles that required repair.

The lower Beacon Hill area from Charles Street to Embankment Road has had a history of problems related to rotted wood piles dating back to 1927. The Boston Inspectional

Services Department (formerly the Building Department) reported that repairs to wood piles had been made at 38 of the 188 houses and buildings in this 10-block area, some having been made in each decade since the 1920s.[17] The Brimmer Street problem described earlier is the most recent example.

Problems in the lower Beacon Hill area appear to be related to both a lowered water table caused primarily by leakage into sewers and drains, and an original wood pile cutoff level at el. 7.0, 2 ft. above the el. 5.0 that was commonly used throughout Back Bay. The area is outside of the Mill Dam and West Side Interceptor and, until 1910, groundwater was readily recharged by the nearby and then tidal Charles River. Construction of the Boston Marginal Conduit and embankment appeared to have impeded groundwater recharge.

Effect of Contemporary Buildings on the Water Table

From the available groundwater data, it is difficult to assess long-term changes in groundwater levels that may have resulted from the construction of buildings in Back Bay. While temporary drawdown has occurred during construction, groundwater has shown that it will return to pre-construction levels unless there is continued pumping from foundation drains or leakage into basements. There is no evidence that buildings constructed in Back Bay within the last 50 years have caused significant permanent adverse effects to groundwater levels. However, older buildings are known to have foundation walls and floors that leak, requiring sump pumping.

Recent heightened public interest in Back Bay groundwater levels prompted an extensive study of existing and probable post-construction groundwater levels around the proposed Hines/New England Mutual Life development at 500 Boylston Street.[18] The study concluded that the building's proposed deep basements would have little impact on long-term off-site groundwater levels.

Preserving Groundwater Levels

The importance of maintaining groundwater levels in Back Bay has been recognized since the late 1800s. Pipes were placed to act as siphons beneath an early sewer and subway tunnel to mitigate their impact on groundwater movement and levels. In several areas, permanent recharge systems have been installed to replenish groundwater, particularly around historic structures founded on wood piles. Temporary recharging around excavations for building construction projects has also been used.

Siphons. In order to lessen the damming effect of the 8-ft. high Boston Marginal Conduit and the wood sheeting that was left in place, Worcester reported that siphon pipes were "placed under the conduit from the Basin to the Back Bay intended to carry groundwater from one side to the other."[15] Worcester questioned the long-term effectiveness of these siphons because they would probably have filled with silt and would have been only locally effective.

Four 12-inch diameter tile siphon pipes were placed under the Boylston Street subway tunnel in the vicinity of Copley Square to transport groundwater from one side to the other. These pipes were probably considered necessary because the bottom of the tunnel was in the clay and its top was at about el. 6.0, thus forming a virtual dam along Boylston Street. The distinct difference in groundwater levels observed on opposite sides of Boylston Street since 1930, confirmed by studies for several projects in recent years, raises doubts about the effectiveness of these siphons.

Siphon pipes were also used in the groundwater equalization system of the recently constructed Southwest Corridor structure to connect perforated header pipes placed on either side of the tunnel.

Recharging. Early inadvertent recharging was undoubtedly performed at many locations by drywells that were used to dispose of precipitation from roofs. These systems were probably not installed frequently enough to have a significant impact on groundwater levels.

In 1930, the first reported recharge system intended to raise groundwater levels in order to protect wood piles was installed at Trinity Church. Only a year before, severely

rotted wood piles were found at the nearby Boston Public Library. Large conductors from the Church's roof gutters were connected into long, stone-filled drywells outside the Church and into a brick-lined pit in the basement. Although dry weather groundwater levels around Trinity Church were below el. 4.0 to 5.0, the intermittent rise in water level and wetting of wood piles due to the recharge system were probably responsible for the preservation of the Church's foundations.

Other recharge systems have since been installed in Copley Square. In the mid-1950s, a recharge system was constructed at a triangular grass plot across Dartmouth Street from the Boston Public Library. In 1968, when Copley Square was redeveloped with its current sunken plaza and fountain, another recharge system was installed below the plaza. Both systems conveyed surface water runoff into the fill through perforated pipes laid in 3 to 5-ft. thick beds of gravel or screened stone.

An underdrain system was provided below the slab-on-grade floor of the Christian Science Church Center parking garage. This system was designed to function only when water levels rose above approximately el. 6.7. The underdrains could be reversed to recharge groundwater should water levels in the fill fall to levels that would threaten wood piles that support the Mother Church. Again, a system of perforated pipes in a thick granular drainage blanket were used.

Recharging to minimize temporary drawdown outside of construction sites had been undertaken for several building projects where there was particular concern for wood piles supporting nearby structures. In these instances, recharging usually involved injecting water, under pressure, into the fill or sand outwash stratum through wellpoints. In some cases, water was pumped into open ditches and large diameter recharge wells and then allowed to percolate into the ground.

Sewer Dams. In the early 1930s, the St. James Avenue sewer was found to be the cause of lowered groundwater levels along most of its length. It was found that when sewage was backed up behind a dam installed in the sewer, groundwater levels rose to "normal" levels, thereby mitigating the effects of leakage into the sewer. Over the years, the original butterfly valves deteriorated and were replaced by a sand bag dam that requires periodic repair.

Since 1985 the MWRA has maintained raised water levels in the Boston Marginal Conduit in order to minimize the impact of local leaking sewers that lower groundwater levels. A permanent solution is still being sought.

Summary

Pumping from water supply wells, accompanied by lowered groundwater levels, has caused subsidence of major cities around the world — including Mexico City, Venice, Taipei and Bangkok. While Boston has not experienced a comparable problem, areas of the Back Bay have suffered damage from lowered groundwater levels. The groundwater table should be restored to levels that preserve the integrity of foundations for the city's historic nineteenth century buildings.

If there were no loss of groundwater by pumping and by leakage into sewers, drains and foundations, and no additions to groundwater from leaking water mains and other man-made sources, the probable groundwater table throughout Back Bay would be expected to vary from el. 8.0 to 10.0. Actual groundwater levels in the fill are lower, except for local groundwater mounds probably caused by leaking water mains. In some areas, the water table is below el. 5.0, a common level at which wood piles were cut off in the nineteenth century.

The principal cause of lowered water levels is leakage into sewers and drains. Groundwater loss through the walls and floors of the Storrow Drive underpass, into subways and the basements of older buildings below the water table also occurs.

With the available data, it is virtually impossible to determine if "permanent" water levels have changed significantly during the past 50 years, except in one or two local areas — for example, the Brimmer Street area and westward along the Boston Marginal Conduit where low water levels have been discovered in the past two years. Little or no water level

data have been available over the past 20 years for major sections of Back Bay, notably the Back Bay Historic District where most buildings are supported on wood piles. A long-term groundwater monitoring program should be established in this and other areas.

The Charles River Basin cannot effectively recharge the groundwater table in the fill because the Mill Dam and Boston Marginal Conduit act as dams. However, some recharging to the sand outwash may occur, but the overall effect throughout the entire Back Bay area is not very significant.

Of the three principal adverse effects of lowered groundwater levels, temporary or permanent, the major future concern in Back Bay is for the deterioration, or rotting, of untreated wood piles. Numerous buildings in Back Bay have suffered damage during the past 60 years from differential settlement caused by rotted wood piles. Problems have been reported in the lower Beacon Hill area in the past two years. Underpinning is currently underway to restore foundations. Future ground subsidence and negative friction on pile foundations are not likely to be significant because of extensive and prolonged dewatering for construction projects over the past 100 years.

There is no evidence that buildings constructed during the past 50 years have caused permanently lowered or significant changes in groundwater levels. Future development in Back Bay would similarly not be expected to cause permanent adverse effects on water levels, provided foundation walls and basement floors are watertight.

Dewatering for the construction of sewers and drains, subways and other transportation corridors, and many buildings has temporarily lowered the groundwater table in the fill and, in particular, the piezometric head in the sand outwash over a large area of Back Bay, in some cases for a period of several years.

Temporary drawdown of piezometric levels in the sand outwash are not likely to cause deterioration of wood piles. There is no evidence of failure or settlement attributed to rotting at the tips of wood piles that commonly bear on the outwash stratum a few feet below the organic silt. Nevertheless, it must be assumed that the sand outwash and fill are connected where construction has penetrated the organic soils. Therefore, dewatering in the sand stratum may affect the groundwater levels in the fill at localized areas that lie a considerable distance from the source of pumping.

Recommendations

In situations where buildings have been constructed with groundwater level sensitive foundations, special emphasis must be given to:

1. An extensive and continuing water table monitoring plan.
2. A thorough review of the temporary and permanent effects of all construction projects on the water table, with construction plans and/or methods altered to minimize their effects on, or replenish, the water table.
3. Rapid response by appropriate agencies to correct lowered groundwater levels found in the monitoring program.

The consequences of repairing and improving water distribution and the sewerage drainage systems throughout Back Bay must be considered. Unless the work is undertaken in the correct sequence, these improvements may adversely affect groundwater levels. For example, since leaking water mains recharge the water table and leaking sewers deplete it, it seems obvious that sewers should be repaired before water mains are fixed. Furthermore, improvements in the sewer system or changes in operations that facilitate drainage and lower fluid levels in pipes will exacerbate the groundwater problem unless leaking pipes are repaired prior to the improvements.

ACKNOWLEDGEMENTS — *This article was originally presented at a meeting of the BSCES Geotechnical Group on March 10, 1986. The authors wish to thank the Boston Redevelopment Authority and Gerald D. Hines Interests for funding the Haley & Aldrich, Inc., study on groundwater in Back Bay Boston, for which the authors were*

principal investigators and upon which much of this article is based.[13] *Local organizations providing data on groundwater levels and information on construction included: Christian Science Church, Trinity Church, Church of the Advent, Boston Public Library, Prudential Center, and the Massachusetts Turnpike Authority. The illustrations were drawn by Ms. Acey Welch and the text was prepared by Ms. Marion Keegan, whose assistance is gratefully acknowledged.*

HARL P. ALDRICH *is Chairman of the Board and a founding principal of Haley & Aldrich, Inc. He received an Sc.D. in civil engineering from MIT in 1951. Dr. Aldrich has over 30 years of experience in solving foundation problems in Back Bay Boston, including recent work on the study of "Groundwater in Back Bay Boston" for the Boston Redevelopment Authority.*

JAMES R. LAMBRECHTS *is a Senior Engineer with Haley & Aldrich, Inc. He received his B.S.C.E. from the University of Maryland in 1973 and an M.S.C.E. from Purdue University in 1976. His experience with the geotechnical problems in Back Bay Boston have been principally associated with the MBTA Southwest Corridor Project. He was also primary investigator for the study of "Groundwater in Back Bay Boston" for the Boston Redevelopment Authority.*

REFERENCES

1. Aldrich, H.P., "Back Bay Boston — Part I," *Journal of the Boston Society of Civil Engineers,* Vol. 57, No. 1, January 1970, pp. 1-33.

2. All elevations used herein are referenced to the Boston City Base (BCB) Datum which is 5.65 ft. below the National Geodetic Vertical Datum (NGVD), formerly called the U.S. Coast and Geodetic Survey *Sea Level Datum of 1929.* El. 0.0 BCB datum is el. -5.65 NGVD.

3. Stearns, F.P., "Report of the Engineer," *Report of the Joint Board Upon the Improvement of Charles River, House No. 775,* April 1894.

4. Cotton, J.E., and Delaney, D.F., "Groundwater Levels on Boston Peninsula, Massachusetts," *Hydrologic Investigations Atlas HA-513,* U.S. Geological Survey, Reston, VA, 1975, 4 sheets.

5. Aldrich, H.P., "Preserving the Foundations of Older Buildings," *Technology & Conservation,* Summer 1979.

6. *Journal of Engineering Societies,* The Boston Society of Civil Engineers, Vol. 1, 1879-1882.

7. Camp, Dresser & McKee, Inc., "Report on Improvements to the Boston Main Drainage System," *HUD Project No. P-Mass-3306 for City of Boston,* September 1967, 2 volumes.

8. Worcester, J.R., "Boston Foundations," *Journal of the Boston Society of Civil Engineers,* Vol. I, No. 1, 1914; with discussions pp. 179-248 and 395-417.

9. Casagrande, A., and Casagrande, L., *Investigation of Settlement at Prudential Center,* Report to Charles Luckman & Associates, September 1970.

10. Casagrande, A., and Avery, S.B., Jr., *Investigation of Building Settlements in Back Bay Area,* Report to Metcalf & Eddy, March 1959.

11. Haley & Aldrich, Inc., *Report on Investigation of Settlement and Lateral Movement, Trinity Church, Boston, MA,* August 1982.

12. Weber, R.P., "Foundation Response Caused by Disturbance of Clay," *Journal of the Geotechnical Engineering Division, ASCE,* Vol. 104, No. GT5, May 1978.

13. Haley & Aldrich, Inc., *Report on Groundwater in Back Bay Boston,* for Boston Redevelopment Authority, Boston, MA, March 1985.

14. Snow, B.F., "Tracing Loss of Groundwater," *Engineering News-Record,* July 2, 1936.

15. Worcester, J.R., & Co., *Report on Pile Foundations and Ground Water Levels at Trinity Church — Boston Public Library - S.S. Pierce Bldgs., Copley Square, Boston, MA,* December 31, 1945.

16. Paine, R.T., *Trinity Church — The Church Endangered by the Low Level of the Ground Water — How the Danger Has Been Temporarily Averted,* April 20, 1935.

17. Folkins, P.M., *Report to the Commissioner of Inspectional Services Department - Structural Report - Lower Beacon Hill, WD 5,* October 15, 1984.

18. Haley & Aldrich, Inc., *Report on Subsurface Investigations and Foundation Design Recommendations, Proposed Development, 500 Boylston Street, Boston, Massachusetts,* for Gerald D. Hines Interests, March 1985.

19. Charles River Basin Commission, *Boston Marginal Conduit: Section 2 — Details of Masonry,* Record Plan Sheet No. 2, May 21, 1910.

20. Charles River Basin Commission, *Boston Marginal Conduit — Section 3, Boston Embankment, Section 1,* Record Plan Sheet No. 8, May 21, 1910.

21. Boston Transit Commission, Plan No. 10478, for Boylston Street Subway, Section 4, December 1912.

A Geotechnical Analysis of the Behavior of the Vaiont Slide

In determining the safety of proposed reservoir slopes, engineers and geologists must have a thorough understanding of the causes of the Vaiont Slide.

A.J. HENDRON, JR., & F.D. PATTON

ON OCTOBER 9, 1963, in the Italian Alps near Longarone, more than 270 million m^3 of rock slid from the side of Mt. Toc at speeds estimated at 20 to 30 m/sec into the newly completed Vaiont Reservoir. Some 2,043 persons died directly as a result of the wave of water that was displaced from the reservoir. The large volume and high velocity of the Vaiont Slide, combined with the great destruction and loss of life that occurred, make it a precedent landslide, particularly for slides caused by reservoir filling. The Vaiont Slide is frequently cited as illustrating one of the hazards that can be caused by dam construction even when the dam is shown to be safe. In fact, the 1963 Vaiont Slide marked a turning point in the amount of emphasis given in hydro projects to the reservoir slopes as compared to the damsite. Major dam projects were delayed or significantly altered in Mexico, Taiwan and Canada, apparently as a direct result of the Vaiont Slide. Modifications were made in many other projects around the world. In the post-Vaiont period, from 1964 to 1967, new regulations concerning reservoirs were introduced in France, Germany, Italy, Japan and the United States, and new recommendations were published by UNESCO.[1]

Engineers and geologists are now generally obliged to examine the slopes of proposed reservoirs. Where unstable slopes are identified, their impact on the project must be described. When the identified slides are large and the effects on the project could be significant, there is an obligation to explain why such slopes are different from and safer than the Vaiont slopes. Such technical evaluations and comparisons require detailed knowledge of the Vaiont Slide, its geology and the geotechnical evaluations made prior to and following the slide. If the engineers cannot give a reasonably complete and consistent explanation of the Vaiont Slide, in terms of currently available methods of stability analyses, then it is difficult to see how they can feel confident about their evaluation of

other reservoir slopes. The disturbing aspect of previous reviews of the Vaiont Slide is that there are gross inconsistencies when the field data, slide behavior and the results of analyses are compared.

The technical literature on Vaiont is abundant, perhaps as a result of the inconsistencies noted. It is likely that more information has been published, and more analyses have been made of the Vaiont data, than for any other slide in the world. However, in spite of this attention, most fundamental questions regarding the failure mechanism and characteristics of the slide have not been satisfactorily explained. For example, an analysis has not been presented that takes into account:

- the obvious three-dimensional shape of the slide surface,
- the actual laboratory shear strengths from representative samples of the material on the slide surface, and
- reasonable piezometric levels related to both rainfall records and reservoir levels.

It is important that a satisfactory set of analyses should take into account these factors and permit the calculation of credible safety factors at various key moments in the history of slide movements in the Vaiont Valley.

In addition, there are many contradictory statements and conclusions in the literature concerning the Vaiont Slide. For example, many authors have claimed, or accepted the claims of others, that there were no significant clays or clayey units present along the failure surface.[2,3,4,5,6] Müller made a point of dismissing the influence of clay interbeds, stating, "Clay and loam, however, were not present in the stratification joints of the Mount Toc, contrary to some publications."[3] Yet, others have tested or described clay beds in the stratigraphic section or attributed them to the failure surface.[7,8,9,10,11,12,13,14,15,16,17]

Another essential factor in an evaluation of the Vaiont Slide is the determination of whether the 1963 slide was a new slide or whether it resulted from the reactivation of a prehistoric slide. Giudici and Semenza mapped and projected the outcrop of a failure surface along the left (south) side of the Vaiont Gorge before the slide occurred.[7] At the same time, they also mapped a unit of an old slide mass on the right (north) side of the gorge near the dam. The existence or absence of an old slide was discussed by Müller and dismissed.[4] He wrote that if one were present, it would not be large enough to be coincident with the actual slide's slip surface.

Objectives of This Study

Finding answers to these questions concerning the clays and the possible existence of an old slide were included as major objectives of this study. These and other questions could be answered by:

- first-hand field observations of the geology,
- an examination of pre-slide and post-slide airphotos,
- laboratory testing of samples of failure plane materials, and
- an examination and translation of geologic and other documents related to pre-slide and post-slide conditions.

Another objective of this study was to perform stability analyses of the Vaiont Slide that were relatively consistent with all the observed facts. Many back-calculations of shear strength parameters for the conditions at failure have been conducted by various investigators on the basis of two-dimensional cross sections. Most of the back-calculated angles of shearing resistance in terms of "effective" stresses (assuming zero cohesion) ranged between 17° and 22°, and several were higher. Even the highest values have been considered by some to be too low.[6]

In those instances where direct shear tests were made on clay materials found in the slide debris, the residual shear strength values of the clays were between 5° and 22°.[15,17] If the clays are moderately continuous, such low values for shear strength as measured in the tests could not readily be reconciled with the results obtained from the analysis of two-dimensional cross sections used by all previous investigators. Calcu-

lations would then show that the slide would be unstable, even without a reservoir, if a shear strength much less than 17° were used. For example, Kenney and Nonveiller back-calculated angles of shearing resistance of 19° to 22° and 17° to 39°, respectively, which were considerable higher than the angles of shearing resistance they had measured on samples from Vaiont.[14,16]

The problem was compounded when the water pressures used in many of the analyses appeared to be too low. This underestimation of water pressures meant that even higher strengths were required if the analyses were to achieve the calculated factors of safety. The enigma was confirmed when the authors, prior to this study, briefly visited Vaiont on two occasions (Patton in 1975 and both authors in 1976). On both occasions, extensive exposures of clay were found along the failure surface. Not only was clay present, but it was a clay with a low angle of shearing resistance.

Additional investigations appeared to be required before analyses could be made that were consistent with all known observations and laboratory data. Such investigations required:

- Direct examination of many locations on the sliding surface to confirm the actual presence or absence of clay
- Obtaining clay samples for shear strength tests, Atterberg limits and clay mineral analyses
- Obtaining a more complete history of the chronology of slide-related events
- Making geological field observations that would determine whether the 1963 event was a first-time slide or the reactivation of an old slide
- Making field observations that would help confirm the actual directions of slide movements
- Defining by field observation any geometrical aspects of the structural geology that would necessitate changes in, or invalidate, the two-dimensional analyses
- Collecting data and making field observations to improve the assessment of water-pressure conditions within the slide

Activities Undertaken

In order to investigate the items listed above, the authors made a one-month field visit to Vaiont during the summer of 1979. During this visit, the slide surface was traversed at numerous locations and extensive samples and measurements were taken. Dr. Edoardo Semenza provided assistance to the authors on pre-slide and post-slide geology. In addition, detailed information on rainfall records, survey displacements, post-slide boring logs and various technical reports were provided by ENEL (Ente Nazionale per l'Energía Elettrica, Compartimento di Venezia).

After the field visit, laboratory tests were conducted on clay samples recovered from the failure plane. Stability analyses were made that utilized detailed knowledge of the three-dimensional structural control of the slide movements and the shear strength data obtained from the clay samples. An analysis of the kinematics of the slide was also conducted and possible mechanisms were investigated that would have resulted in the loss of strength necessary for the slide to have moved to the position observed on the opposite valley wall. These kinematic studies are presented in a previous paper by the authors prepared with the assistance of Anderson.[18,50]

Description of the Slide

The Vaiont Slide is located east of Longarone, which is on the Piave River some 100 km north of Venice (see Figure 1). The slide developed along the north slopes of Mt. Toc where the Vaiont River had cut a canyon more than 300 m deep just above its junction with the Piave River.

The slide moved a 250 m-thick mass of rock (approximately 270 million m^3) some 300 to 400 m horizontally with an estimated velocity of 20 to 30 m/sec before running up and stopping against the opposite side of the Vaiont Valley wall. The new slide displaced an old slide mass that had been isolated on the north side of the valley. The old slide materials moved some 100 to 150 m above their original position before slumping backwards 30 to 40 m to the south.[12] The uppermost por-

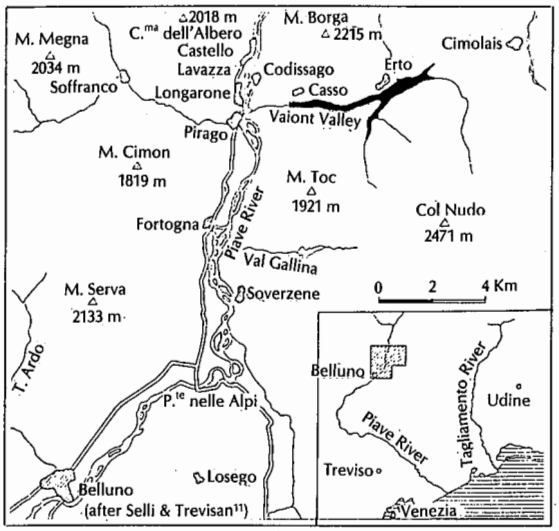


FIGURE 1. Location of the Vaiont Valley, Italy, approximately 100 km north of Venice, and northwest of Udine.

tion of the eastern half of the slide apparently moved over the main slide mass in a separate and slightly later movement. In a matter of a few tens of seconds, the slide filled the lower half of the Vaiont Reservoir (which had been drawn down to elevation 700 m from a level of 710 m just prior to the slide).

The wave resulting from the displaced water propagated both upstream and downstream. The wave eroded trees and soil on the north side of the Vaiont Valley up to a maximum elevation of 935 m (235 m above the reservoir level). The wave swept across the dam, reaching over 100 m above its crest (435 m above the downstream base of the dam), and moved down the Vaiont Gorge. The wave had a height of some 70 m at the confluence with the Piave River and it destroyed most of the town of Longarone and parts of other towns in the Piave Valley (see Figure 2). Some 2,043 persons died and many others were injured, almost all from the effects of the wave. Most of the loss of life occurred in Longarone, but the loss was also severe in nearby villages, especially Pirago. Forty-five men, who were part of a work force of engineers, technicians and laborers living in barracks on the dam crest, were killed. Over $16 million was reported paid for civil suits for personal injury and loss of life. Tens of millions of dollars of property damage resulted. The $100 million dam and reservoir were abandoned. The destruction associated with the Vaiont Slide and wave have been described by many authors.[10,19,20,21,22,23,24] The Vaiont Slide was a major tragedy of the 1960s.

If both volume and velocity of slide movement are considered, the Vaiont Slide has been exceeded in historic times in only a few cases, such as the 1974 Mantaro Slide in Peru and the 1911 Pamir Slide in the USSR. However, other slides of greater volume than the Vaiont Slide have been recognized, and

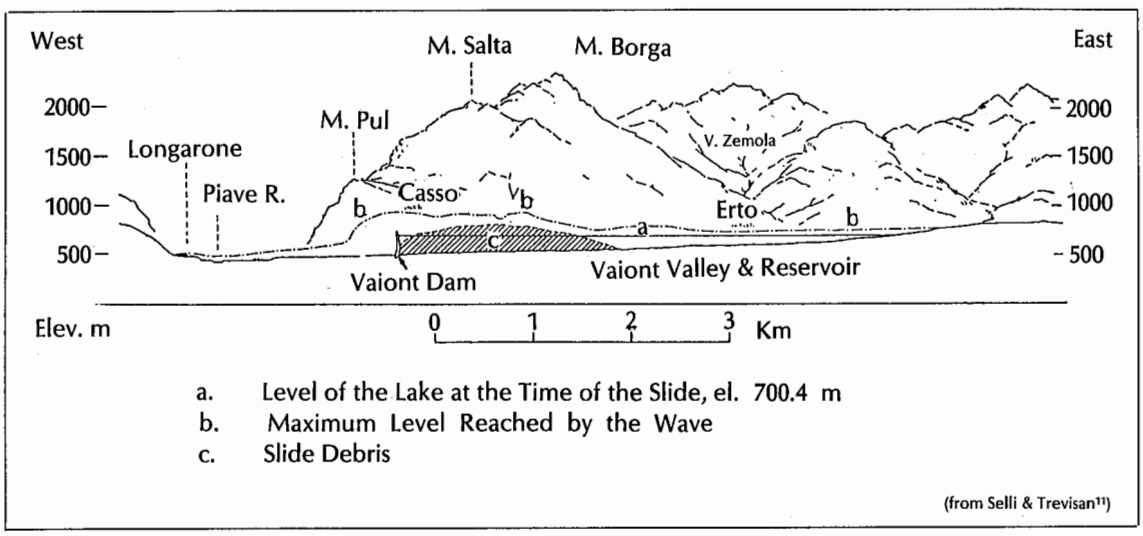


FIGURE 2. Longitudinal profile along the Vaiont Valley, looking north.

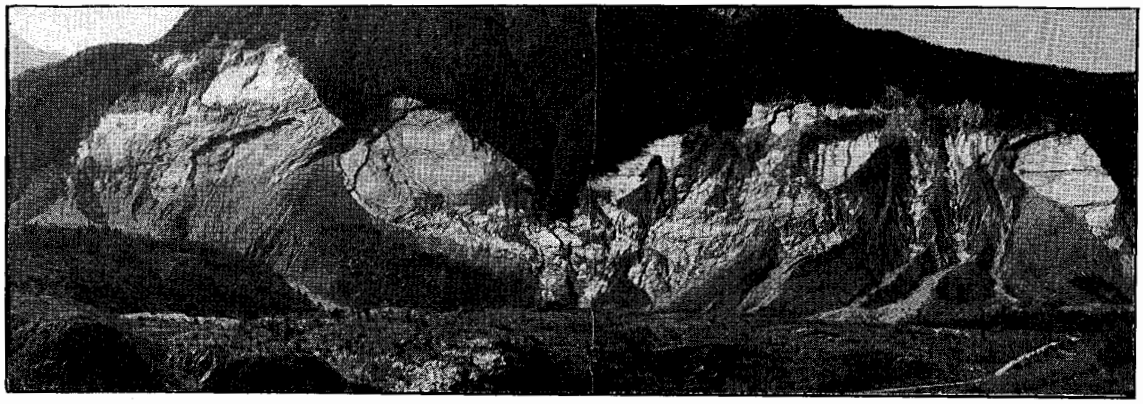


FIGURE 3. Photograph of the Vaiont Slide area, July 1979.

many higher-velocity slides of smaller volume are known.

A photograph of the Vaiont Slide as it appeared in July 1979 is given in Figure 3. This photograph shows the M-shaped outline of the slide as seen when facing Mt. Toc from Casso, north of the slide. A plan view of the immediate area of the slide prior to October 9, 1963, is given in Figure 4. The plan view shows the slide in relation to the Vaiont Dam and the maximum proposed reservoir level (el. 722.5 m). Prior to the slide, the principal feature in this area was the central north-south trending dry valley of the Massalezza, a tributary to the Vaiont River. This valley has been referred to as the Massalezza Ditch. On the left side of the Vaiont River, and just downstream from the junction with the Massalezza Ditch, was a prominent bluff at el. 777 m called the Punta del Toc. A prominent bench at about el. 840 to 850 m was present part-way up the western side of the slide. This plain was called the Pian della Pozza, or Pozza, and contained several enclosed depressions similar to those found in karstic or glaciated regions, or in areas with old landslide debris. Along the toe of the slide, the Vaiont River varied in elevation from 500 m near the dam to 560 m at the upstream side of the slide.

A few people lived on what became the slide area, but the closest town was Casso, perched above a cliff opposite the slide at about el. 940 to 980 m (see Figures 2 and 4). The lowest two buildings in Casso were damaged by water or by the air blast generated by the wave. The remainder of Casso escaped damage. The larger town of Erto, located on the north side of the Vaiont Valley some 3.5 km upstream from the dam and 1.5 km from the slide mass, escaped heavy damage from the wave since the town is over 760 m elevation. The wave at Erto reached about 740 m in elevation (40 m above the reservoir).

Previous Geologic Studies

The starting point for detailed geologic studies of the Vaiont Slide is the 1960 report, prepared in 1959-1960, by Giudici and Semenza.[7] Following the 1963 slide, many other geologic studies were made for one or the other of the investigative commissions and for the ENEL, the hydroelectric authority that had taken over control of the project prior to the slide. Of these post-slide studies, Semenza's is particularly helpful since it provides a history of the geological and geophysical studies from 1959 to 1964.[12]

Following the slide, other important geological studies were published.[2,4,9,11,20,25,26,27,28] Of these studies, only Broili's and Müller's reports are in English.[2,4,20] Kiersch wrote a brief summary in English providing an early account of the slide, its causes and associated flooding as well as the general geologic features noted following the slide.[10] Because of the timely nature of Kiersch's article, it received widespread attention in North America. The collected works of Selli and Trevisan, Carloni and Mazzanti, and Ciabatti constitute essential documents on the geology, slide observations, seismic data and dynamic eva-

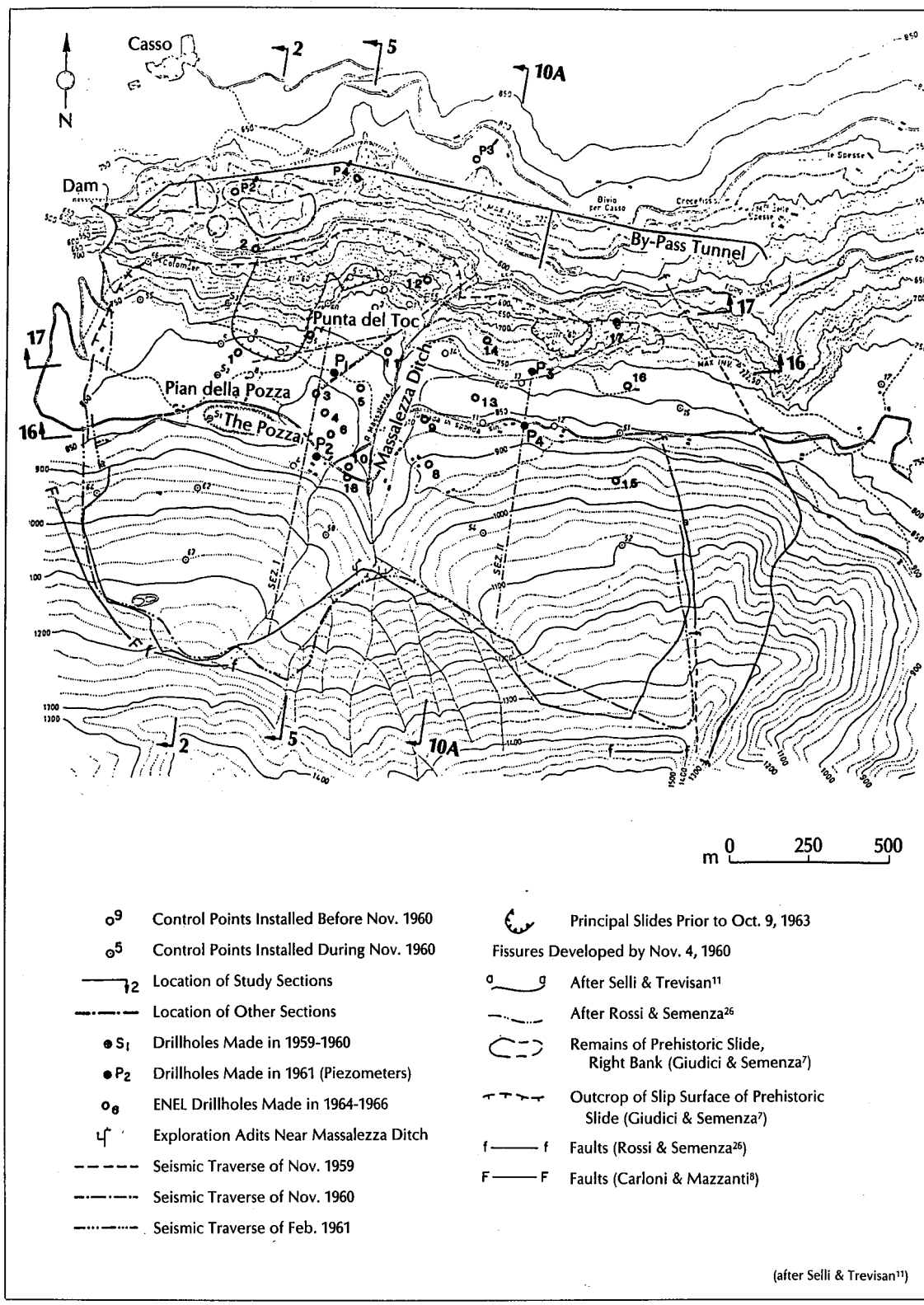


FIGURE 4. Plan view of the valley prior to the slide of October 9, 1963.

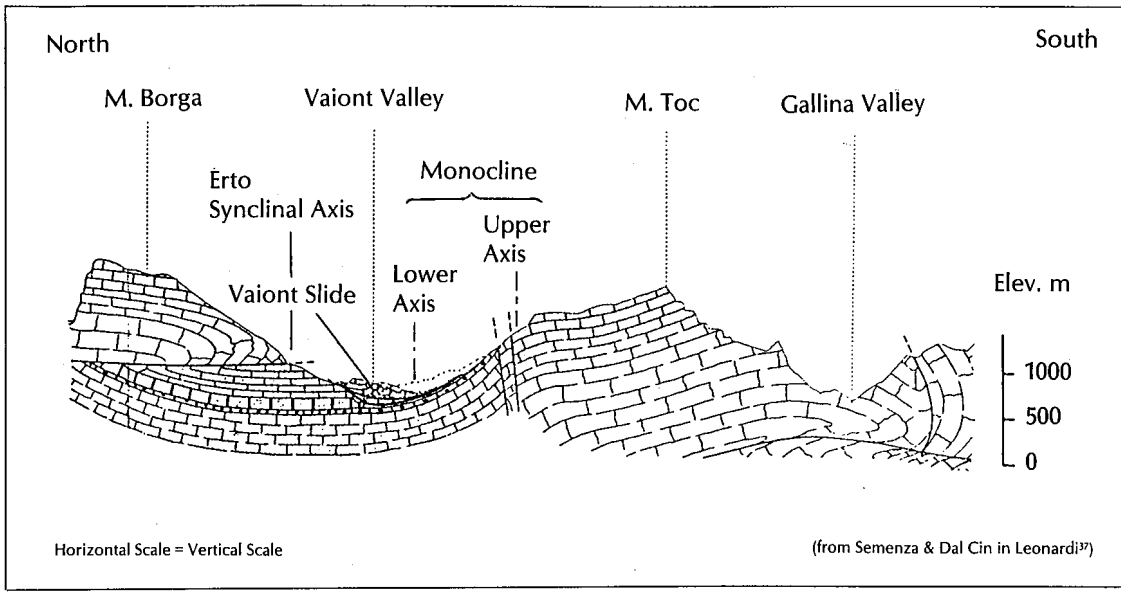


FIGURE 5. Regional north-south geologic section through the Vaiont Slide.

luation of the slide.[9,11,29]

The original geologic mapping of the Vaiont Valley was undertaken by Boyer.[30] Boyer prepared a cross section from Mt. Toc to Mt. Borga across the Vaiont Valley in the vicinity of the Vaiont Slide. However, his section does not indicate an ancestral slide. Müller reports that a geologic study of the reservoir sides conducted by Dal Piaz did not indicate any wall movements.[20,31] The regional geology of the Vaiont area has been studied in more recent times by Rossi and Semenza, Semenza, Leonardi and Semenza, and others.[26,32,33,34,35,36] Many of these studies are contained within the beautifully illustrated two-volume compendium on the geology of the Dolomites, *Le Dolomiti,* edited by Leonardi.[37]

General Geologic Setting

The Vaiont Slide is located in the southeastern part of the Dolomite Region of the Italian Alps. The mountains in this area are characterized by massive near-vertical cliffs formed by the Jurassic Dogger formation and underlying Triassic formations. The local valleys tend to be associated with outcrops of the weaker formations, particularly the Upper Jurassic, Cretaceous and Tertiary units that contain more clays and are more thinly bedded.

The Vaiont Valley has been eroded along the axis of an east-west trending, asymmetrical syncline plunging upstream to the east. This feature has been called the *Erto syncline.* The syncline is shown extending under Mt. Borga north of the slide on Figure 5. The upstream plunge is shown in Figure 6 on a section made by Broili through the toe of the slide.[2]

An abrupt monoclinal flexure on the south limb of the Erto syncline forms a distinctive and important aspect of the geology of the slide. The axis of the lower fold of the monocline is aligned subparallel to the Vaiont River, some 400 to 800 m to the north (see Figure 5). Between this axis, which forms the rear of the "seat of the chair," and the river there is a 9° to 20° eastward dip of the beds down the plunge of the syncline. South of this axis, the beds dip to the north towards the Vaiont River at 25° to 45°. These beds form the "back" of the slide. The axis of the upper fold of the monocline corresponds to the top of the head scarp of the western portion of the slide. This upper axis is shown in Figure 5 but does not show up on the sections shown in Figure 7 on page 74, since the sections do not extend far enough south.

Most sections of the slide presented in the literature have been drawn down the

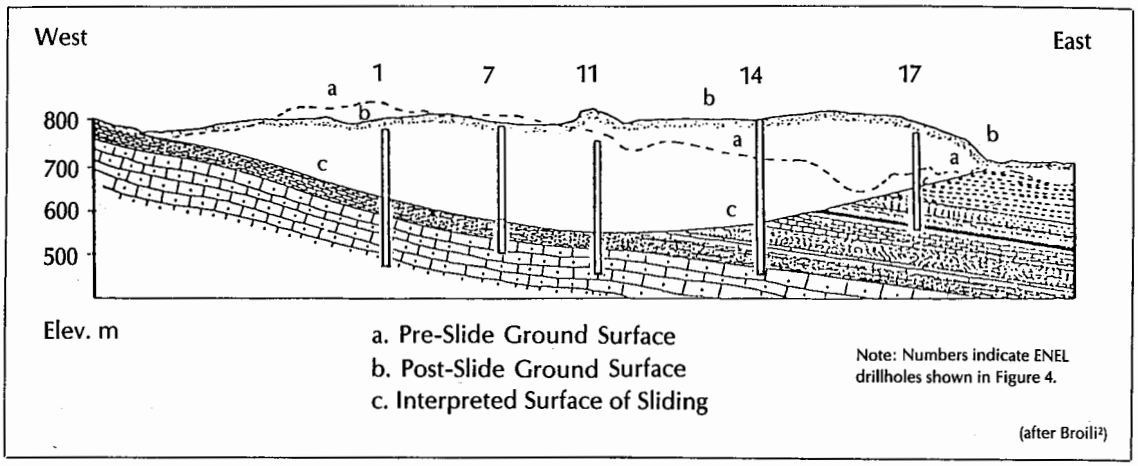


FIGURE 6. East-west geologic section through the toe of the slide.

maximum dip of the steeper exposed bedding planes located at the "back" of the slide. A number of these sections are depicted in Figure 7. Several of these sections show the flat apparent dip of the "seat" of the slide that, although it is reasonable for these sections, is misleading because the true dip of the beds along the seat is 9° to 22° to the east as noted above. The easterly plunge of the beds forming the Erto syncline had a significant effect on the behavior of the slide.

Evidence was found suggesting that the 1963 surface of sliding had a complex origin and corresponded with more than one previous period of rupture. These periods include both a prehistoric landslide, or landslides, and possibly a much older period of tectonic faulting. The seat of these different periods of shearing displacements were the weak clay interbeds in the Malm and Lower Cretaceous units. Evidence of the tectonic faulting observed in this study includes widely scattered outcrops of a cemented breccia and one occurrence of fault-like grooves. The outcrop of these previous rupture surfaces prior to 1963 corresponds with the one shown on Figure 4 as mapped by Giudici and Semenza along the left side of the Vaiont River gorge and described by Semenza.[7,12] The elevation of this plane varies from 700 m near the dam to 540 m about 1 km east of the dam. From there, the plain rises slowly to el. 650 to 660 m in the next 700 m upstream to join the east side of the slide. This rupture surface generally tends to follow the bedding planes on the downstream (western) side of the slide, but appears to cut across or step up to successively higher clay interbeds on the upstream (eastern) side of the slide. It seems unlikely that the surface of sliding would cut smoothly across the bedding as shown by Broili on Figure 6. The eastern side of the slide appears to follow, in part, a fault that was oriented roughly perpendicular to the river (see Figure 4). Faults have also been mapped along the headscarp of the slide, and some have been mapped along the western side of the slide.

The Vaiont Valley has an extremely deep and narrow inner gorge some 300 m deep that was eroded within a broader glaciated valley.[9] Following deglaciation, a predecessor to the present Vaiont Canyon was eroded into the syncline, thereby releasing one or more prehistoric rock slides. Part of one of these slides buried alluvium that was infilling a deep bedrock channel, possibly a pre-glacial valley. This channel and the overlying slide mass were first mapped by Giudici and Semenza (see Figure 7a).[7] After these early events, the present canyon was eroded at the site of the Vaiont Reservoir. The present canyon appears to have resulted from downcutting and erosion of the river through an old slide mass that had originated from the south side of the valley. This process set the stage for a repeat performance of the prehistoric slide. There is evidence that movements

of the old slide on the south side of the Vaiont River occurred as the canyon was being eroded to its 1963 pre-slide configuration.

General Stratigraphy

The succession of stratigraphic units in the Vaiont Valley has been the subject of many reports and published papers. The first study containing details of the stratigraphy relevant to the slide included a brief stratigraphic sequence based on a review of literature and field work in the summer of 1959 and spring of 1960, describing the Jurassic-Cretaceous-Eocene sequence of rocks present.[7] The principal units were described in greater detail by Semenza and his stratigraphic description is presented on pages 86-87.

The bedrock in the slide area consists of a thick succession of limestone and marly limestone beds of Upper Jurassic and Lower and Upper Cretaceous ages. Brecciated limestones are present, frequently with chert nodules, in addition to lesser amounts of dolomites. Some of the local limestone and dolomite beds have a high porosity due to solution features. Clay interbeds are reported to be particularly common in the Upper Jurassic rocks. A simplified and independent description of the local stratigraphic units of the rocks exposed in the slide area is presented in Figure 8a.[9] The base of the Vaiont Slide lies within the Lower Cretaceous (the c_1 unit of Carloni and Mazzanti and the *a* unit of Semenza[9,12]) and within the Upper Jurassic Malm (unit g_3 of Carloni, unit *ma* of Semenza) that overlies the oolitic beds of the Dogger formation. The thickness of the beds at the base of the slide averages about 5 to 10 cm, but varies from 1 to 20 cm. However, the Dogger limestone (see Figure 8b), which lies a short distance below the failure plane, is "massive," with the thickness of the beds generally exceeding 0.5 to 1.0 m.

Clay Interbeds & Layers

The most significant aspects of the stratigraphy are the location, continuity and physical properties of the clay interbeds in the rock column. This topic has been a controversial one and was the subject of an extensive report and technical paper by Broili on work undertaken at the direction of L. Müller at the Institute for Soil Mechanics and Rock Mechanics, Karlsruhe.[2] Broili's work was based on a review of the core logs obtained from drillholes that were made for a study conducted by ENEL after the slide. The micropaleontological and petrographic studies of these cores were undertaken by G.A. Venzo and A. Fuganti of the Geology Institute of Trieste University. Further studies of geologic sections in the slide area were also made by these geologists. Broili concluded that "the succession does not include any clay beds or intercalations which some authors consider may have been responsible for some aspects of the phenomenon" (p. 80).[2] Broili's work was cited by Müller to support his contention that "contrary to several publications, no clay existed on the slip surface."[4]

Consequently, the authors were surprised, during preliminary examinations of the failure surface in 1975 and 1976, to note extensive clay interbeds and layers of clay intimately associated with the surface of the 1963 slide. In July 1979 the authors, accompanied by H.R. Smith and G. Fernandez, had further opportunity to examine and sample the exposed portions of the failure surfaces during a three-week period. The locations of these observations and clay samples are shown in Figure 9 and are described in detail in Hendron and Patton.[18] A summary of the field observations of the clay layers is presented in Table 1 on pages 80-81.

Not all of the failure surfaces examined had resulted from the October 9, 1963, slide. Many of the visible rock faces were formed by later slides involving slabs of rock that "broke down" to one of the many adjacent underlying clay interbeds.

Where the surface of sliding is overlain by slide debris, the clays were generally found preserved (for example, see Figures 10, 11 and 12). However, where the failure plane has been exposed, the clays are rapidly eroded by rainfall and by debris flows from the large catchment surfaces (see Figures 13 and 14). Small folds and faulted monoclinal (cascade) structures, which are present in many areas of the slide but are not visible at a

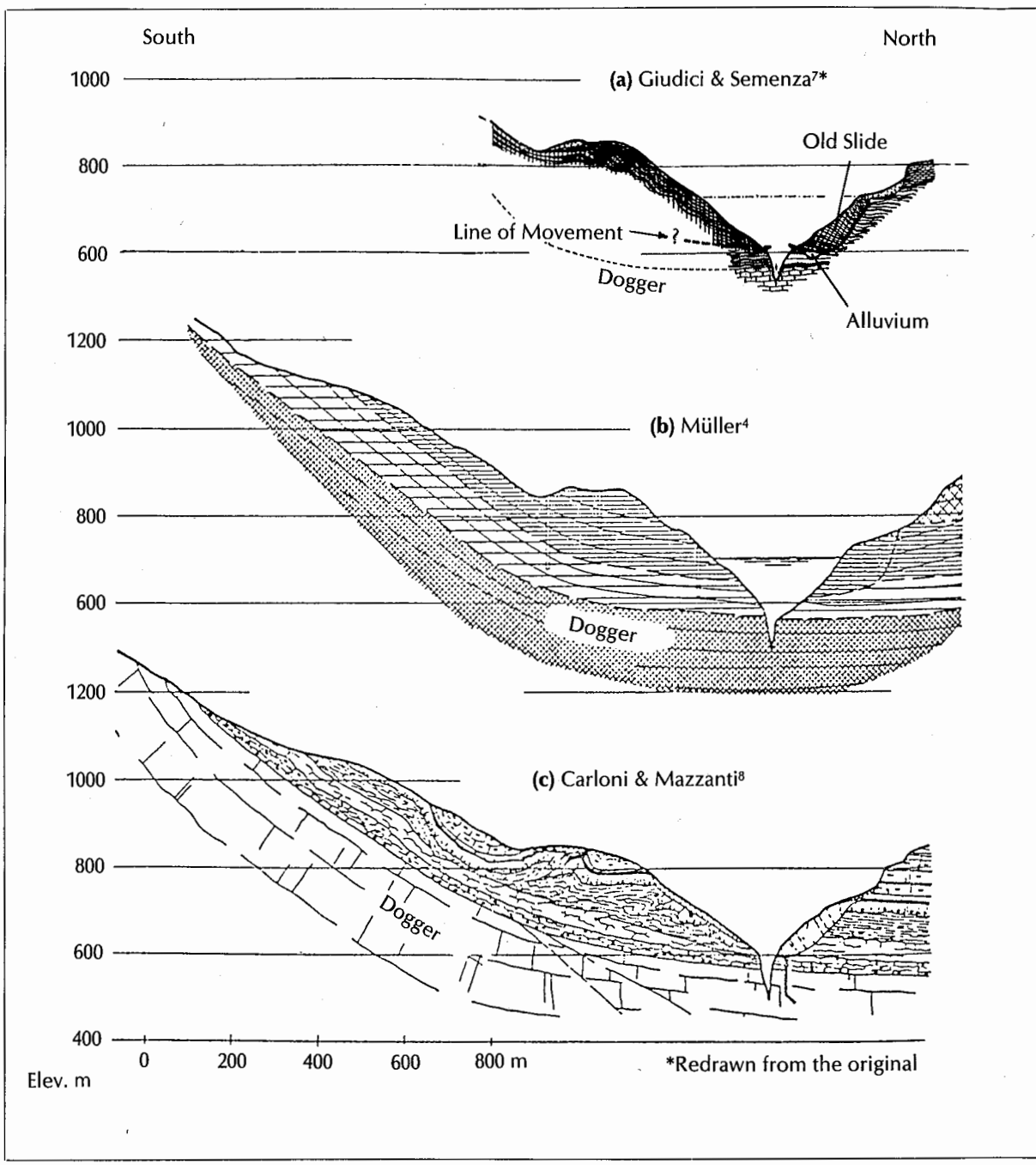
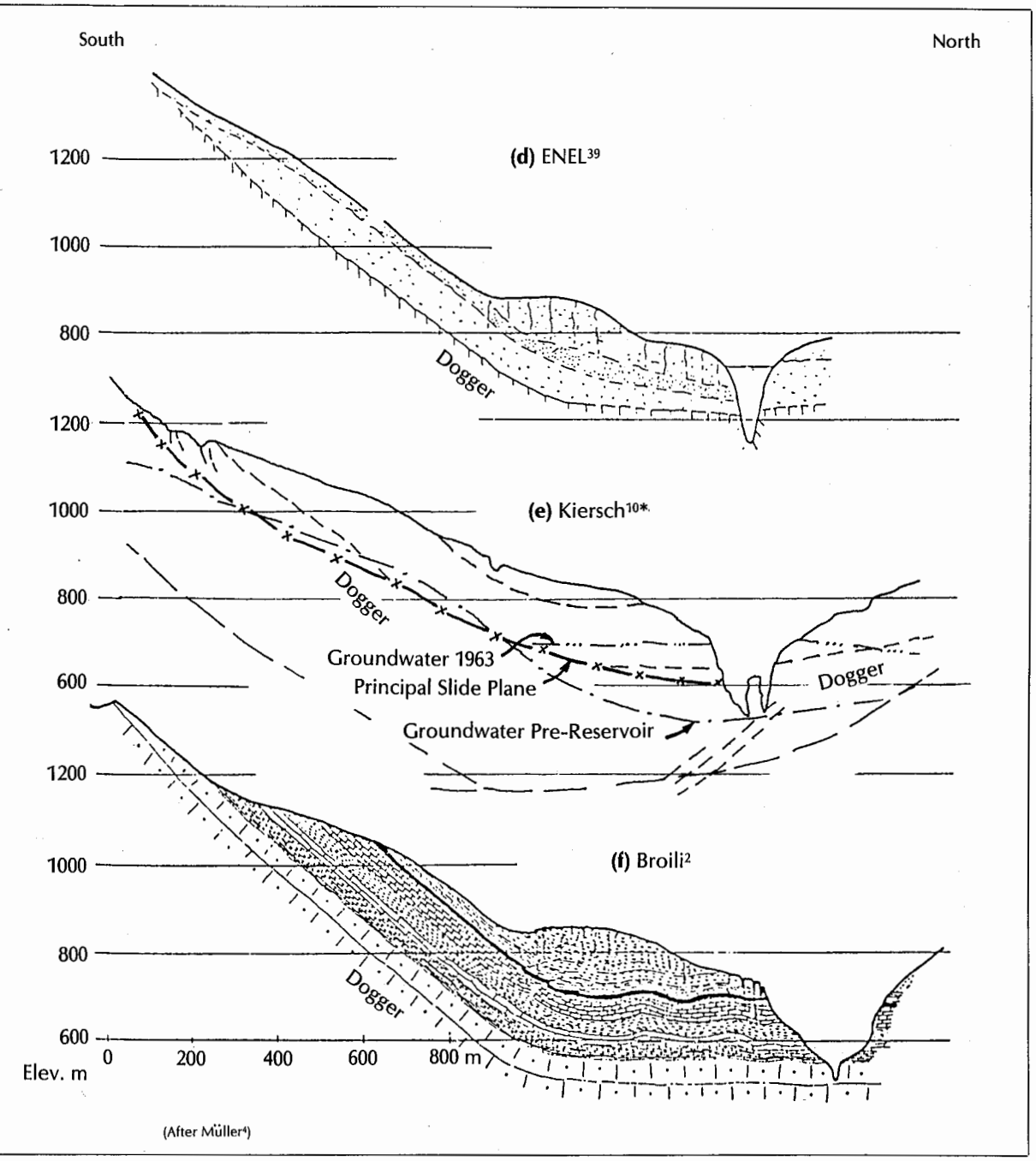


FIGURE 7. Geologic sections of six other investigators.

distance (Figure 15), have served to protect and preserve small portions of the clay interbeds that are strategically continuous with large adjacent areas of the 1963 sliding surface (for example, see Figure 16).

The lower 10 to 30 m portion of slide debris exposed along the base of the rock outcrops on the west side of the slide consists of an uncemented angular gravel and sand-sized breccia with frequent layers of clay and breccia with a clay matrix (for example, see Figure 17). The clay layers in this breccia fre-



quently exhibit structures that suggest shearing of the upper layers over the lower layers. Although the clays are often mixed with angular breccia, layers and interbeds of clay without noticeable sand-sized particles were observed with thicknesses of 10 to 15 cm, with occasional greater thicknesses. The clay layers in the breccia are commonly 1 to 4 cm thick. Lumps of clay were reported on the surface of the slide by Nonveiller, who tested the strength of one of these lumps.[16] Similar lumps were found by the authors at numerous locations on the slide debris surface. Clay layers along the surface of sliding of the 1963 slide are commonly 1 or 2 cm thick, but vary from 0.5 to 10 cm or more.

(a) Description of the Stratigraphic Column

Upper Cretaceous

- c_8 Marly limestones, silty, pink-colored. *Scaglia Fm* beds 50 cm thick, total thickness 300 m.
- c_7 Limestones, red-colored, *basal Scaglia Fm*, total thickness approx. 15-20 m.
- c_6 Cherty limestones, grayish to reddish, nodular beds 5-200 cm, interbeds of gray-green marly limestone to marls, age *Turonian & Lower Senonian*, total thickness approx. 100 m.
- c_5 Marly limestone, pink & red, age *Cenoman*, total thickness 1.5 m.
- c_4 Limestone with some green clayey interbeds, age *Cenoman*, total thickness 3-4 m.
- c_3 Marl & marly limestone, pink, age *Cenoman*, thickness 3-4 m (weak unit).
- c_2 Brecciated limestone & marly limestones, beds 10-100 cm thick, slump structures, age *Albian*, total thickness 10-20 cm.

Hiatus

Lower Cretaceous

- c_1 Marly limestone, pink & green color 5-30 cm thick, nodulars of dark chert, clastic limestones at top. Some green clay or marly limestone beds, age *Albian*, total thickness 45-60 m (weak unit).

Upper Jurassic to Lower Cretaceous

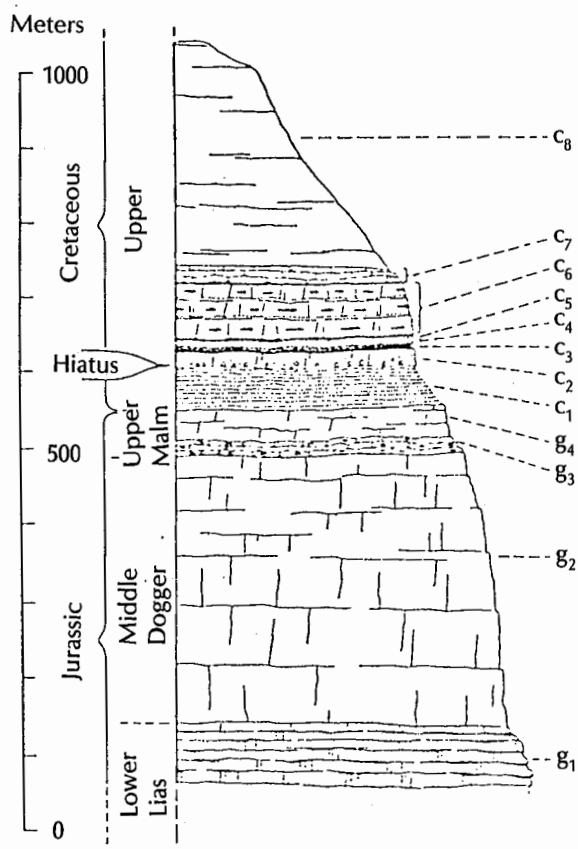
- g_4 Compact limestone, grayish to reddish color sometimes with chert nodules, beds 30-40 cm thick (1 m thick in lower 20 m), age *Upper Malm to Lower Cretaceous*, total thickness 40-45 m (weak unit?).

Middle Jurassic

- g_3 Cherty limestone, dark gray color, beds 5-20 cm thick, nodular reddish chert, age *Malm*, total thickness 25-35 m (weak unit?).
- g_2 Oolitic limestones to dolomitic limestones, locally porous dolomite due to solution, beds in upper part 0.5-1 m thick, otherwise approx. 1 m, age *Dogger*, total thickness 350 m.
- g_1 Limestone, gray to bluish well-stratified, beds 5-15 cm thick, partings of bituminous marl, age *Lias*, total thickness 80-100 m.

(after Carloni & Mazzanti, & Broili[8,2])

(b) Geologic Column, Vaiont Valley



(after Carloni & Mazzanti[8])

FIGURE 8. Geologic column of the valley, with a description of the column.

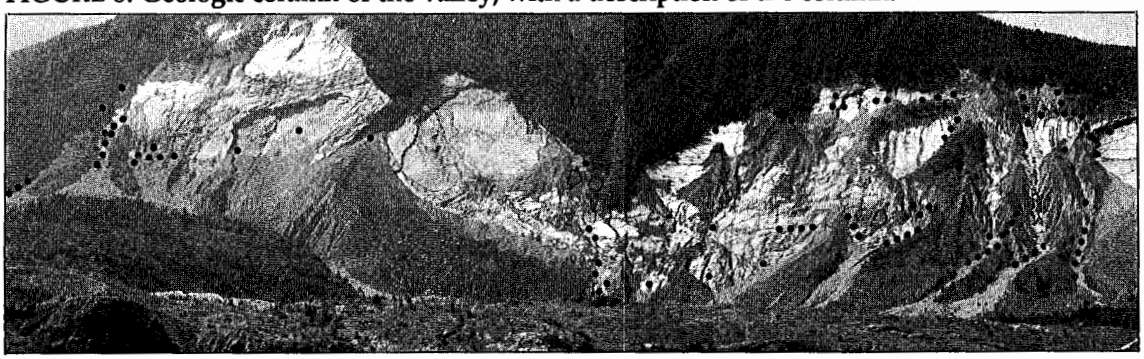


FIGURE 9. Photograph of the Vaiont Slide showing locations of field observations. Clay layers were observed at most locations. An arrow and open circle indicate the location of one of the exploratory adits.

FIGURE 10. A clay layer 5 to 20 cm thick along the failure surface at location 12-3, near the east side of the slide.

When exposed in the field in a destressed condition, the clay is generally very soft and sticky and has a slight "popcorn" or cracked and fluffy surface because it has been subjected to frequent wetting and drying cycles. Such characteristics are typical of montmorillonitic clays. The presence of the clay layers in the field can often be inferred from the presence of small slumps whose failure surface corresponds to one of the clay layers. When these slumps are trenched and examined, the soft, sticky clays can be readily identified. The dry clay fragments slake rapidly in fresh water. When the clay interbeds remain in their original stratigraphic position within the undeformed bedrock, the material is much firmer. The thickness and frequency of clay interbeds seemed to diminish with increasing distance below the bedrock-slide debris contact. Thick layers of clay were found in the slide mass and at the contact with the underlying bedrock surface. In isolated areas of partly displaced slide debris at the top of the slide, clays were found indicating that at least one layer of clay occurred several meters above the surface of sliding of the 1963 slide that was thicker than any found at the base of the 1963 slide.

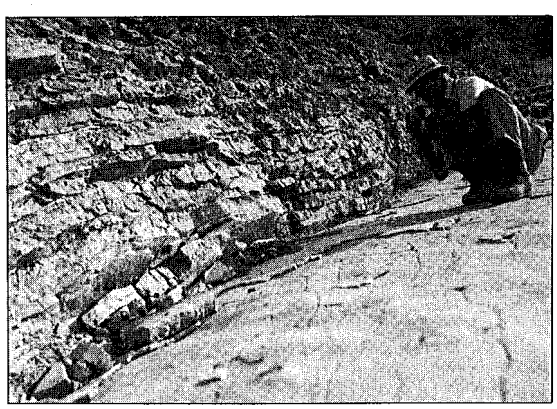


FIGURE 11. A view of the failure surface below the headscarp at location 18-9, about at the western third point of the slide.

FIGURE 12. Clay interbed 5 cm thick along the failure surface at location 18-9. Other thin clay interbeds up to 0.5 cm thick lie in the rock mass.

Evidence of the stratigraphic continuity of the clay interbeds found in the slide was sought away from the slide, particularly in the

FIGURE 13. Slide debris overlies the failure surface where a clay layer 4 to 6 cm thick has been eroded from beneath the debris at location 11-7, the lower portion of the fourth gully from the west side of the slide.

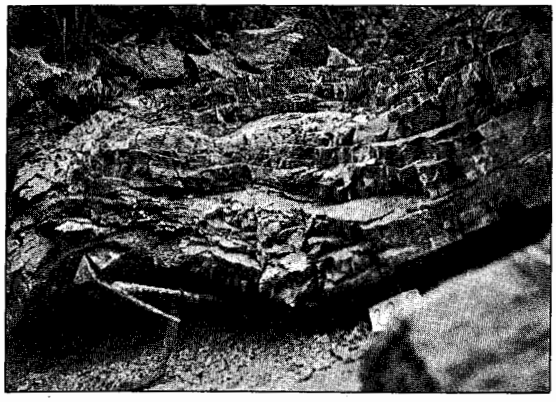


FIGURE 14. A view of a clay interbed 7 to 10 cm thick protected by a small fold and associated with adjacent failure surfaces at location 22-8, the fifth gully from the west side of the slide, one-third the way up the rock face.

valley of the Mesazzo Torrent just east of the slide on the slopes above the dam on the north side of the Vaiont Valley south and west of Casso. At this location, a series of five continuous clay interbeds varying from 0.5 to 17.5 cm thick was located within 20 to 30 m of the same stratigraphic position as the surface of sliding of the Vaiont Slide. Outcrop 8-1 is near Casso and is shown in Figure 18. This outcrop lies just above the main path leading to Casso from the west at about el. 940 m. Samples taken from these clay interbeds have similar Atterberg limits (presented in Table 2

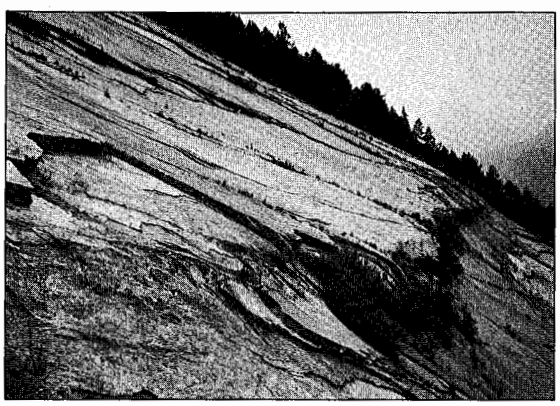


FIGURE 15. A monocline located in the middle of the western rock face. The surface, exposed after the October 1963 slide, is associated with a clay interbed 0.2 to 1 cm thick.

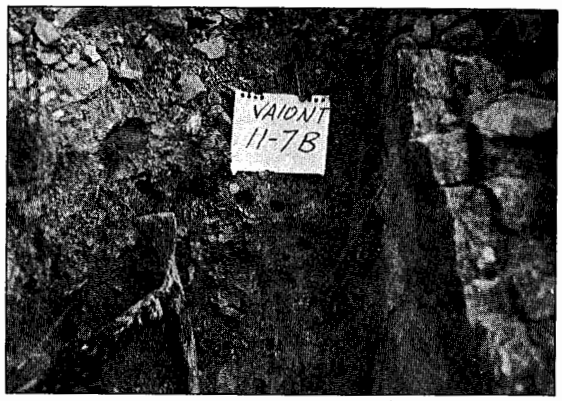


FIGURE 16. A clay layer, at location 11-7B, 10 cm thick lying between bedrock to the right and slide debris to the left. The clay is protected by a cascade structure, but is associated with the nearby failure surface.

on pages 82-83) to those taken from the failure surface of the slide. A sketch of this outcrop is shown in Figure 19.

The evidence from outcrop 8-1 indicates that clay interbeds are characteristic of the rock units that correspond to those forming the base of the slide. Such stratigraphic clay units would be expected to be continuous over substantial areas and would not have "originated during the sliding movement of the rock masses along the slip surface" as was concluded by Broili (p.80).[2]

Some of the confusion concerning the

FIGURE 17. Five clay layers 1 to 2 cm thick within the lower 1.5 m of slide debris at location 11-10, the fourth gully from the west side of the slide. The failure surface appears in the lower left corner.

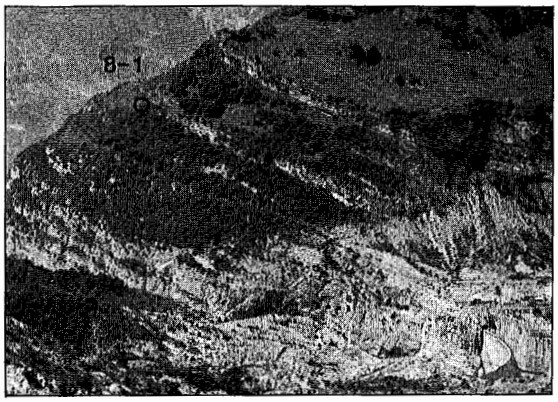


FIGURE 18. Location 8-1 where in-situ clay layers outcrop southwest of Casso in the same stratigraphic horizon as the failure surface. The top of the Vaiont Dam is in the lower right.

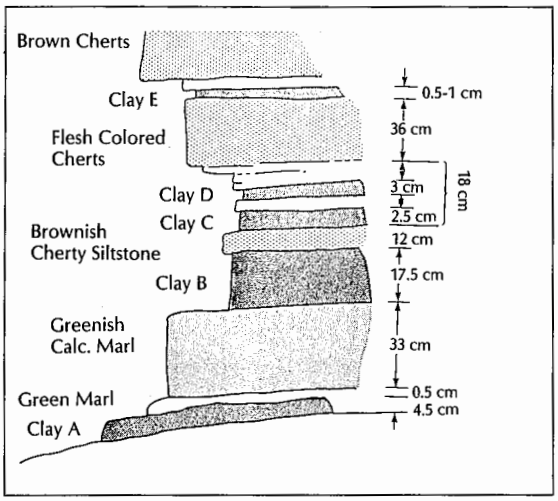


FIGURE 19. A sketch of the outcrop of Malm rocks southwest of Casso. This outcrop lies in the same stratigraphic sequence as those at the base of the Vaiont Slide.

clay layers seems to result from differences in terminology. Broili summarized some of the different descriptions and terminology.[2] Giudici and Semenza wrote, in reference to the Lower Cretaceous rocks, that "numerous intercalations of greenish clay, with thicknesses of a few centimeters, are present."[7] Kiersch mentions clay seams, claystone interbeds, marl and clay partings in the Malm and Lower Cretaceous beds.[10] Other descriptions of the clays mention "mylonitic" and "ultramylonitic facies."[12] Martinis described "reddish or greenish calcareous marls in the form of streaks or extremely thin layers" and "limestone interbedded with greenish foliated marls."[25] Others have described the clayey materials as "thin films of pelitic material."

Any clay bed in a folded stratigraphic sequence of alternating hard and softer units will be subjected to differential shearing displacements along bedding planes due to the flexural-slips as described by Skempton, and Patton and Deere.[13,38] Therefore, a sheared and slickensided structure would be expected in portions of all such clay beds. It seems to be of little consequence with respect to the slide to argue whether the layers are clay, pelite, argillite, foliated marl, clayey marl, marly clay, soft calcareous marl, biomictite or largely argillaceous. All such materials, when sheared, are likely to result in an uncemented clay-rich slickensided material.

Numerous stratigraphically continuous layers of uncemented clay-rich materials are present. The clay content varies from: 16 percent,[4] to 35 to 38 percent montmorillonite,[2] to 50 to 80 percent.[15,18] Because the predominant clay mineral is a calcium montmorillonite, all of the preceding percentages are sufficient to produce soil mixtures that have very low values of the residual angle of shearing resistance.

Broili, after his study of the core from post-slide drillholes, seemed to dismiss the influence of clay along the surface of sliding.[2] However, the core recovery was very poor (0 to 20 percent) in the lower Cretaceous materials, and often remained poor (20 to 30 percent) in the Malm and Dogger units below the failure surface. Under these circumstances of low core recovery, little clay was recovered from the drill core. Furthermore, for the first few years after the slide, many of the excellent outcrops now present were covered by the slide material. Thus, the number and thickness of the clay layers and interbeds may have been difficult to ascertain immediately after the slide.

During this study the geologic logs of the ENEL holes drilled after the slide were examined. The ENEL boring P'-2 (not piezometer P2), located on the north side of the valley

TABLE 1

Summary of Observations in Clay Layers & Related Features

Location No.	Clay layer noted in slide debris no. & thickness, cm	Clay layer on 1963 failure surface no. & thickness, cm	Other clay layers, + = above 1963 surface - = below 1963 surface no. & thickness, cm
9-1	1,—	—	—
9-2	—	—	-1, 0.2-1
9-3	—	—	(no thickness noted)
9-3A	—	possibly	—, 0.2-0.5
9-3B	—	—	—
9-4	—	1, 1-2	—
9-5	—	1, —	—
10-1	1, —	—	—
10-2	—	1, 2	—
10-2A	1, 1-2	1, 0.5-2	—
10-3	—	1, —	—
10-3A	6 layers, 1-10	1, 1-10	—
10-4	Several	1, 2-4+	-2 cm, 1-2
10-4A	Several	1, 1-2	—
10-4B	Several	1, 1-2	—
10-5	1, —	1, 4-6	—
10-6	—	1, 1-3	±5 layers, 0.4-1.5
10-6A	—	1, 1-3	+5 layers
11-1	>5 layers in 1.5 m debris	—	—
	~12 m clay rich debris	—	—
11-2	—	—	-6 to 10 layers
11-2B	—	1, 0.5-1.0	—
11-3	Debris has clay matrix	—	—
	1, -1 cm w/slick	1,1-2 w/slick	—
11-4	—	1, 1-6	—
11-4A	—	1, 0.1-5	—
11-5	1 - 1 m	—	—
11-6	1, —	—	—
11-7	—	1, 2-6	-2, 4-6
11-7A	1, 0.3-0.5 (just above f.p.)	—	—
11-7B	—	1, 10	-1, 0.2
11-8A	—	1, 0.5-2	—
11-9	1, —	1, 2-10	—
11-10	4 layers (very clay-rich)	1, 1-2	—
11-10A	2, 1 & 15	failure plane at base of thickest layer	—
12-1	—	—	-1, 0.1-0.5
12-1A	—	—	—
12-1B	—	—	—
12-2	2, 20 cm breccia with clay matrix	1, 2	—
12-2A	—	—	—
12-3	1, 50 cm clay rich matrix	1, 5-20	—
12-3A	2, —	—	—
12-4	—	1, 0.1-1.5	—
12-4A	—	1, 0.2-0.5	-3, 0.2-0.5
12-5	—	1, 2-3	many, 0.2-0.4
12-6	1, 15	1, 1-2	—
12-7	1, 15	—	—
18-2	1, —	1, 2-4	—
18-3	—	contact not visible	—
18-4	—	contact not visible	—
18-5	—	contact not visible	—
18-6	—	no clays left on failure surface (rock-debris-rock contact)	—

Location No.	Clay layer noted in slide debris no. & thickness, cm	Clay layer on 1963 failure surface no. & thickness, cm	Other clay layers, + = above 1963 surface - = below 1963 surface no. & thickness, cm
18-6A	—	—	—
18-7	—	—	—
18-8	—	1, 5-10	-1, 3
18-9	—	1, 5	+2, trace-0.5
18-10	—	1, 2	—
18-11	—	1, 2	—
18-14	1, 5	1, 2-10	—
22-1	—	no clay visible (cascade structure)	—
22-1A	1, several cm discontinuous	1, trace discontinuous	—
22-2	—	1, 1-4	-1 to 6 layers
22-3	1, —	1, 4-10	-2, trace
22-3A	1, —	1, 2-5 x 8 m +length	many, 0.5-8
22-4	—	1, 0-5	—
22-5	—	no clays visible (removed by erosion)	—
22-6	—	1, 1-3	—
22-6A	—	1, 1-4	—
22-7	—	1, 7-10	—
22-7A	—	1, 1-2	—
22-7B	—	1, 2-6	—
22-8	—	1, 2-10	+1, 1-3
23-1	—	no clays visible some buckling of rock slabs	—
23-2	—	—	-3 layers, 2-1
23-3	—	1, 8	—
23-4	—	1, 8-10	—
23-10	—	1, 2-3	-2 layers, 1-3 & 10-15
23-11	—	1, 2-3	—
23-12	—	1, 1-5	—
23-13	—	portal of old adit	—
23-14	—	top of Dogger (no clays visible)	—
23-15	—	no clays visible in Dogger Form (fault in Dogger)	—
23-16	—	no clay readily visible (access difficult)	—
23-17	—	old failure plane (no excavation for clays made) no clay visible 1-2 m cover	—
24-1	several, 1	1, 0.5-1	—
24-2	—	1, 0-6	—
24-3	—	1, 2-4	several, 1
24-3A	—	1, 2-3	—
24-4	—	1, 2-4	—
24-6	(on failure plane of post 10/9/63 slide)	—	-3 layers, 0.5
24-7	—	2, 0.5-1.5	—
24-8	—	—	-1, 1
69-1	—	1, 2-3	—
69-3	—	1, 2-5	—
69-4	—	1, 5	—
67-1	1, —	—	—
67-2	1, —	—	—
522-2	1, —	—	—
522-3	1, —	—	—
522-4	—	1, —	—
522-5	—	1, 1-6 x 80 m long	—
522-5A	—	2 layers	2-10
522-6	—	1, 0.2-1	—
522-7	—	—	1, 1-3

TABLE 2

Atterberg Limits on Clay Samples

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Descriptive Notes
8-1	67	28	39	In-situ clay, same unit as base of slide
8-1A	80	35	45	In-situ clay, same unit as base of slide
8-1B	68	36	31	In-situ clay, same unit as base of slide
8-1C	50	30	20	In-situ clay, same unit as base of slide
8-1D	72	29	43	In-situ clay, same unit as base of slide
9-1	76	32	44	Clay in slide debris
9-3A	33	20	13	In-situ clay sample on failure plane
9-5	58	21	37	Clay on failure plane
10-2	52	30	22	Clay at rock-debris contact
10-2A	53	32	21	Same as 10-2 (4 m away)
10-3A	68	35	33	Lower 1 m of debris above failure plane
10-4	39	24	15	Clay at rock-debris contact
10-4A	40	24	16	Clay at rock-debris contact (8 m from 10-4)
10-6	38	26	12	Clay at rock-debris contact in-situ
11-1	70	21	49	Clay in debris 50 m from rock contact
11-2A	66	33	33	Clay at rock-debris contact
11-3	56	32	24	Clay layer 1-2 cm above failure plane
11-4	50	27	23	Clay layer at rock-debris contact (10 m from 11-3)
11-5A	92	36	56	Clay layer just above debris (8 m from 11-4)
11-6	55	31	24	Large clay block, float in slide debris
11-7B	61	26	36	Clay layer just (1-2 cm) above failure plane
11-8	48	27	21	Clay at rock-debris contact
11-9	76	26	50	Clay at failure plane (2-10 cm thick)
11-9	67	30	37	Direct shear tests by WES
11-10	76	36	40	Clay at failure plane, 4 layers (1-10 cm thick) in debris above sample
12-1	26	16	10	Clay silt layer, east scarp
12-2	72	22	50	Clay at slide debris-tectonic breccia contact
12-3	76	22	54	Clay layer at debris-tectonic breccia contact
12-4	73	29	44	Clay in-situ in failure plane
12-5	56	29	27	Clay in-situ on main failure surface over east side of slide

some 450 m upstream from the right abutment of the dam, encountered a series of layers of brecciated debris. In some of these layers, clay was noted. These clayey layers varied from 1 to 3 m in thickness. These layers are in the stratigraphic position of the extension of the basal rupture plane of Giudici and Semenza;[7] the area was mapped as old slide material by Rossi and Semenza as shown in Figures 4 and 7a.[26]

Although the core recovery was extremely poor in most other post-slide drillholes made in the slide debris, the following observations are noted in the ENEL logs drilled after the slide:

- Drillhole 7 encountered 5 m of reddish clayey rock fragments just above the in-situ rock
- Drillhole 8 encountered detritus with

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Descriptive Notes
12-6	72	23	49	Clay in debris about 4 m above failure plane
12-6A	35	19	16	Clay on failure plane (10 m from 12-6)
18-6	49	27	22	Clay on failure plain at scarp
18-6A	39	20	19	Clayey debris on rock surface near scarp
18-8	45	32	13	Clay in-situ forms failure plane above
18-9	37	25	12	Clay in-situ forms adjacent failure plane
18-9A	48	33	15	Clay in-situ on failure plane
18-11	38	25	13	Clay in-situ forms failure plane below
18-14	43	30	13	Clay layer in debris above failure plane
22-1A	57	20	37	Clay layer on failure plane
22-3	42	14	28	Clay layer between slide debris & tectonic breccia
22-3A	50	25	25	Clay in base of debris just above tectonic breccia
22-4	54	32	22	Clay below cemented breccia on bedrock contact
22-6A	44	25	19	Clay layer, in-situ, forms failure plane above
22-7	48	26	22	Clay at rock-debris contact
22-7B	37	28	9	Clay layer, in-situ below failure plane
22-8	37	25	12	Clay layer, in-situ
23-3	46	32	14	Clay layer, in-situ, forms failure plane above
23-4	60	33	27	Clay layer between debris & rock
23-10	57	30	27	Clay layer, in-situ
23-11	57	35	22	Clay layer, in-situ forms adjacent failure plane
23-12	46	28	18	Clay layer, in-situ forms adjacent failure plane
23-17B	39	21	18	Clay layer, in-situ in fold
24-1	68	31	37	Clay layer with cemented breccia
24-2	82	22	60	Upper clay layer in cemented breccia
24-2A	64	32	32	Lower clay layer in cemented breccia
24-3	45	23	22	Clay layer along failure plane
24-7	39	22	17	Clay layer, in-situ, in fold
25-3	55	30	25	Clay layer, in-situ in Malm
522-5A	66	23	43	Clay layer on failure plane
522-5A	81	24	57	Clay layer on failure plane

clay in the lower 7 m of the slide mass

- Drillhole 9 encountered 31.5 m of clayey debris
- Drillhole 11 encountered two zones of "argilla" (clay) with detritus, at depths of 48 to 71 m and 91 to 107 m
- Drillhole 13 encountered 4 m of clay with rock fragments 0.7 m above the base of the slide mass
- Drillhole 18 encountered clayey debris near the base of the slide, and further encountered 3.5 m of calcareous chert intercalated with clay just above the in-situ rock

Therefore, it would appear that there is considerable evidence of clay and clayey debris at the base of the slide mass, in spite of the fact that soft clay with rock fragments can be very difficult to recover from drillholes.

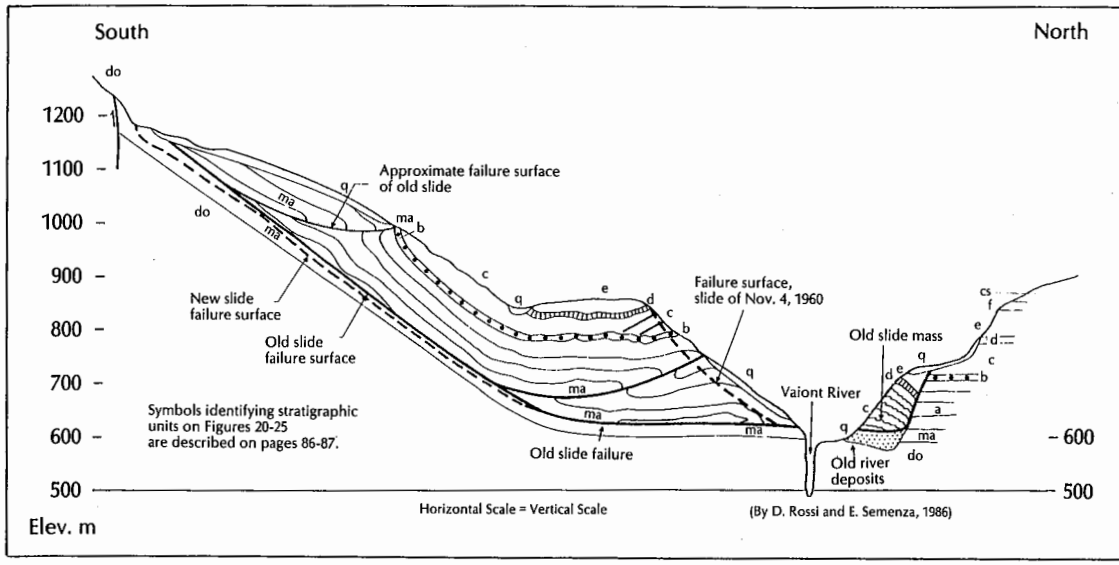


FIGURE 20. Geologic Section 2 before October 9, 1963.

Multiple layers of weak clays were present along much of the surface of sliding. These clays are largely stratigraphic in origin, although undoubtedly some shearing and development of slickensides had occurred prior to the sliding activity. This conclusion is not in agreement with the conclusions given in the principal technical papers on the slide in English.[2,4,20] However, this is in agreement with a conclusion of the Frattini Commission that noted:[39]

> "Yet, in the material accumulated by the slide we can see clay beds, a few centimeters thick, separated by small or less flinty, nodular calcareous strata.
>
> In our opinion, these strata, of a really clayey nature, cannot be considered the product of sliding; they may rather be of sedimentary origin... The Malm and the base of the Lower Cretaceous, which are calcareous-nodular, with flint nodules or beds and clay interstrata, forms a mass that can easily be deformed, minutely cracked and subject to cataclasis."

Structural Geology

The basic structures affecting the slide are:

- The steep back of the slide that provided the driving forces,
- The pronounced eastward dip of the seat of the slide,
- The continuous layers of very weak clays within the bedded rocks, and
- The faults along the eastern boundary of the slide.

Giudici and Semenza mapped the outcrop of a prehistoric failure surface along the Vaiont Canyon walls (see Figure 4), showed its limits at both ends and indicated that the entire area was a zone of possible sliding.[7] On their section (shown in Figure 7), they showed no uphill limit to the base of this zone that ended in question marks. However, a simple extension of their projected "line of movement," indicating the base of the slide, would extend to or beyond the depression of the Pozza. The steep back and flat toe (on a north-south section) of the slide was established by their mapping and interpretation of the Dogger-Malm contact. From their geologic map, the upstream dip of the failure surface along the walls of the canyon could be determined. They had also established the existence of a block of old slide material on the right abutment and stated that it had come from the south side of the valley in a previous slide. The outcrop of the failure surface mapped by Giudici and Semenza is essentially coincident with the 1963 surface of

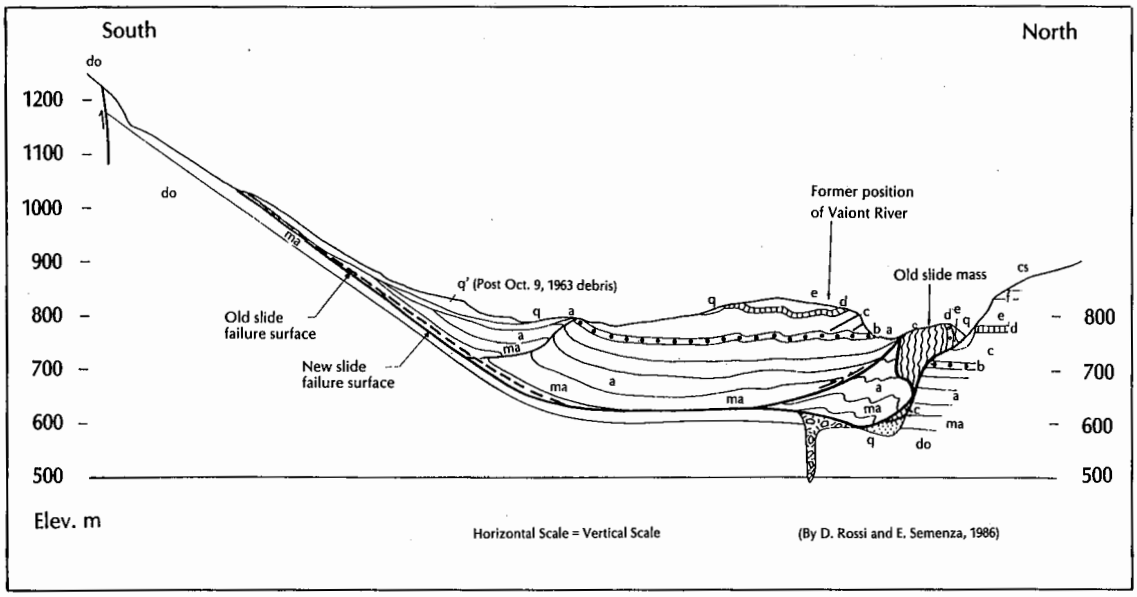


FIGURE 21. Geologic Section 2 after October 9, 1963.

sliding.

Two geologic maps prepared by Rossi and Semenza provide an accurate and detailed picture of the geologic structures present before and after the slide.[26] (These maps were reproduced as Figures 11 and 12 in Hendron and Patton.[18])

In the course of this study, particular interest was paid to Sections 2, 5 and 10A in Figure 4. These sections were selected as representative sections for use in stability analyses. They were also chosen because they appeared to be oriented relatively close to the direction of the original movement of the slide. At the request of the authors, Rossi and Semenza undertook to interpret the geology along these sections both before and after the 1963 slide. Their interpretations of Sections 2, 5 and 10A are presented as Figures 20 to 25. The symbols used for the units in these sections are given on pages 86-87.

Figure 20 depicts Section 2 before the October 9, 1963, slide. Two different minor variations in the interpreted surface of sliding are shown along the steeply inclined portion of the slide. Figure 20 also indicates a fault at the top of the slide area and some previous sliding within the future slide mass. A portion of the old remnant block of slide material on the right-hand side of the Vaiont Valley is also shown. The ground surface after the slide of November 4, 1960, is depicted by the dashed surface above the canyon wall. This surface approximates the surface of sliding of this precursor slide.

Figure 21 portrays Section 2 after October 9, 1963. It indicates a moderately simple downhill translation of the slide. The figure also depicts a remarkable upward displacement of the old slide material on the right-hand valley wall and some of the post October 9, 1963, debris and alluvial fans covering portions of the surface of the main mass. Figure 21 shows that the failure plane is parallel to the bedding in the upper part of the slide, but cuts across the bedding in the lower part of the back of the slide. The authors agree with the general position and orientation of the slide surface shown for the exposed portions of the failure surface. Whether the surface of sliding cuts across beds at depth or not is a matter of interpretation. Appreciably more drillholes would be required to better define the degree of conformity of the failure surface to the bedding.

Figure 22 presents Section 5 before October 9, 1963. It illustrates a section with a large stabilizing toe relative to the small volume that acts as a driving force.

Figure 23 shows Section 5 after October

A Description of the Stratigraphic Units Associated With the Vaiont Slide

This description of the stratigraphic units associated with the Vaiont Slide is taken from a translation of Semenza[12] that has been published as Appendix G in Hendron and Patton.[18] The following sequence can be observed proceeding from the oldest to the most recent formations.

do **Dogger:** oolitic and crystalline limestones. The Dogger is a very compact and extremely rigid formation, poorly stratified in thick layers, intensely fractured and very permeable. The formation is about 300 m thick. The dam lies on this formation. In addition, the Dogger formation is the basal structure of the slide zone. The Dogger outcrops on the Mt. Toc slopes above the area from which the landslide moved as well as to the west of that area and in the Vaiont gorge, below the dam. This formation was not involved in the movement.

ma **Malm:** gray cherty limestones with black cherts, which can be nodular. The Malm formation is composed of very thin strata (not more than 15 cm) with abundant interlacing of cherty material or scattered cherty nodules. The Malm has some interbedding of thin calcareous sheets or soft marly-calcareous materials. It is easily fractured or folded, but the Malm is much more compact than the formation just above it. The overall thickness of the Malm cannot be precisely evaluated, but ranges from 30 to 50 m. The Malm formation outcrops at the top of the Costa delle Ortiche and at other points along the slope of Mt. Toc beyond the surfaces where the landslide broke away. It can also be observed at the right side of the Vaiont Valley above the dam. (Note: Recent work by Semenza and Rossi to be presented this year has led them to conclude that the surface of sliding is mainly within the Malm.)

a **Lower and Middle Cretaceous** (lower part ≃ Aptian). This formation is formed of a complex of limestones or marly limestones, containing cherts, with thin soft calcareous, marly or clayey-marly interbeds. The color is prevalently red in the upper part, greenish in the middle and light gray at the base. The formation consists of intensely fractured thin strata and, on the whole, is rather easily deformable. Its thickness is 120 m and can only be estimated along the fault wall coinciding with the eastern boundary where the mass broke away. In addition to this area, the formation appears on slabs remaining along the rupture surface. In fact, the surface of sliding corresponds to many wide tracts of different strata of the lower part of complex *a*, which are joined by almost vertical cuts perpendicular to the strata. Complex *a* may be found in various places in the slide mass, usually at the peripheral zones, but also, in particular, in the zone of the craters near the dam, and in the eastern lobe.

b **Middle Cretaceous:** (middle-upper part) conglomerate with pinkish or gray cement. This unit forms a bed about 10 m thick, which is topped by a calcareous layer about 1 m thick. It can be distinguished from the conglomerate of level *d* because the cement uniting the fragments is pinkish or gray rather than white. This extremely compact conglomerate stands out from other formations, frequently forming a step. It is visible in numerous points; in particular along the western flank of the Colle Isolato, at the base of the Pinnacolo and on the southwestern side of the Conca delle Pozza.

c **Middle-upper Cretaceous** (Albian ≃Cenomanian). This complex was originally

9, 1963. This figure depicts almost complete removal of the activating forces after the slide. Just after the slide, Semenza discovered evidence of a 30 to 40 m southern movement of the mass after it had reached its maximum northern limit.[12] This backward movement is

formed of rather compact beds of gray limestones that alternated with sequences of less resistant thin layers of greenish limestones and calcareous marls. On the whole, the original permeability probably was quite low; today the permeability is high due to intensive fracturing. This sequence may be easily observed on the Pinnacolo and at the southwest edge of the northwest wall of Punta del Toc.

d **Upper Cretaceous** (≃Turonian). This complex consists of red marly, silty limestones interbedded with conglomerates and limestones. Three distinct and characteristic units appear in this complex: the upper and lower units are red, and the intermediate unit is often conglomeritic, sometimes revealing syngenetic folds. All three units together are about 14 m thick. On the whole, they have low strength characteristics. This complex is found in many parts of the slide mass, in particular along the northwest and north walls of the Punta del Toc, and along both sides of the Massalezza Valley.

e **Upper Cretaceous** (≃Coniacian). This complex consists of fine-grained limestones with various colored cherts. It is about 27 m thick, formed of limestones of various types, rich in cherts and showing a prevalently nodular structure at the base. Originally, this complex must have been extremely strong and compact, even now its strength is greater than other horizons, as may be seen on the north wall of Punta del Toc and on the plateaus of the Pozza and east of the Massalezza where this cherty limestone outcrops extensively.

f **Upper Cretaceous** (≃Santonian). This is a complex of light red and green limestones and marls with red cherts. The general pinkish coloration of this complex is lighter than that of the levels described above. It is, however, more compact. This complex outcrops primarily in the northeastern zone of the slide mass.

cs **Scaglia rossa** of the Upper Cretaceous. The scaglia rossa consists of marly-limestones at its base and of marls in the remainder. These marls are generally red except for a gray intercalation. This formation was not involved in the movement.

q **Quaternary.** This unit consists of deposits older than the landslide — specifically, morainic, detrital and alluvial. For the main part, they consist of coarse detrital material containing somewhat rounded elements. This material is abundant in the northeastern zone. A limited area of lacustrine clays is visible on the Pozza plateau. The presence of morainic deposits remains problematic.

q o **Detritus:** deposited by the wave produced by the landslide. This unit consists of detritus of various origins stripped away from detrital or alluvial slopes or torn from outcrops of intensely fractured rock by the force of the wave. These materials are easily recognized because they show no cementation, compaction or settlement. The form assumed by these deposits permits a reconstruction of the movement of the wave. They are widely distributed over the area, especially in the lowest zones.

q d **Detrital masses and alluvial fans:** formed after the landslide. This material consists of detritus of rock slides that slid from the slabs exposed along the slide surface and of alluvial cones that formed primarily during the rains of November 1963. In various places, these materials have considerably modified the topography to the point that it no longer corresponds to the topographical conditions noted immediately after the slide. This applies, in particular, to the internal lake that was almost half-filled by such detrital materials.

portrayed by the two directions of movement noted on the planes of sliding at the toe. A comparison of Figures 22 and 23 reveals only modest changes in the structure of the majority of the displaced rock mass other than translational and rotational movements.

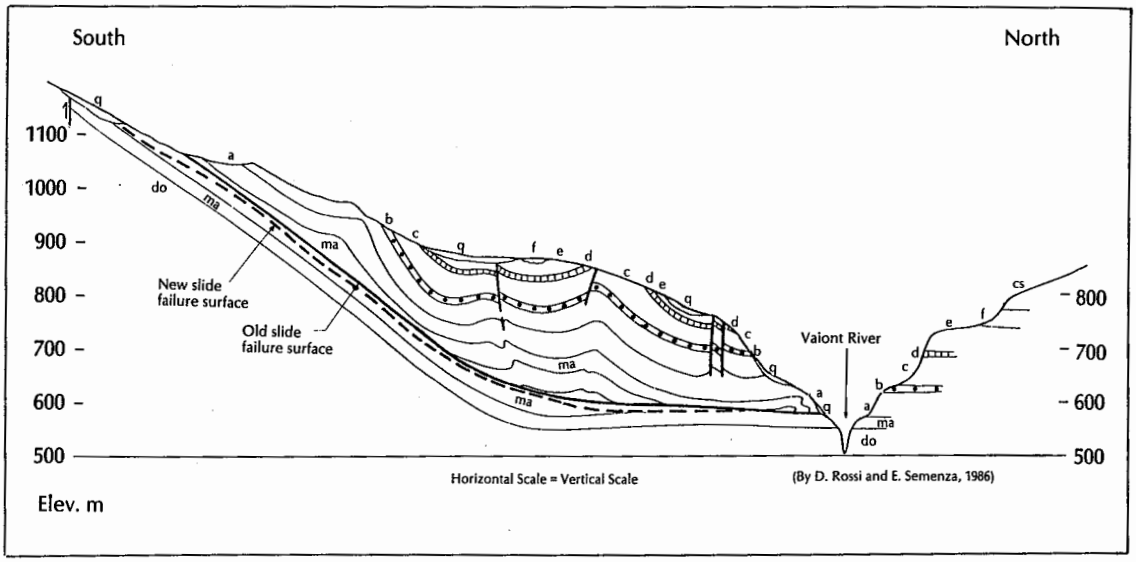


FIGURE 22. Geologic Section 5 before October 9, 1963.

Geologic Section 10A presented in Figure 24 shows the geologic structure of a representative section of the eastern side of the slide taken in the approximate direction of initial movement of the main slide mass. Most sections of this part of the slide presented in previous investigations have been oriented some 10° to 20° counterclockwise in plan view to give more emphasis to the direction of movement of the top of the eastern portion of the slide. Rossi and Semenza called this upper part of the slide the Eastern Lobe in Figures 24 and 25, and show it in the area A-B before the slide. Post-slide surface depositional features suggest that the Eastern Lobe did not start its movement until after the main slide mass had completed most of its movement. The two slide movements appear to be essentially independent and the Eastern Lobe appears to have followed the movement of the main slide and formed the slide material shown in the area A-B on Figure 25. Rossi and

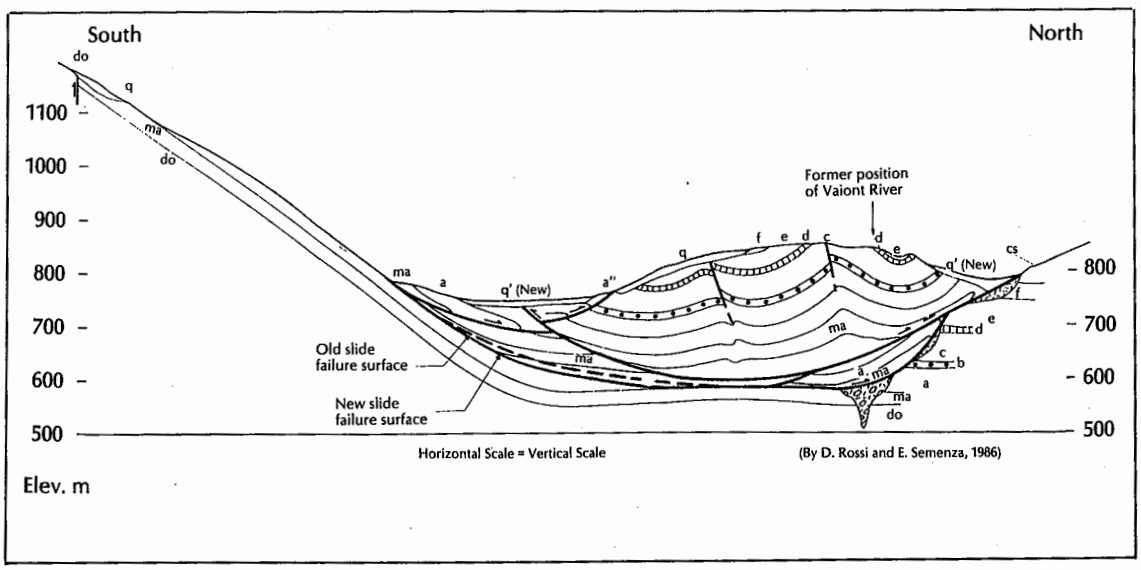


FIGURE 23. Geologic Section 5 after October 9, 1963.

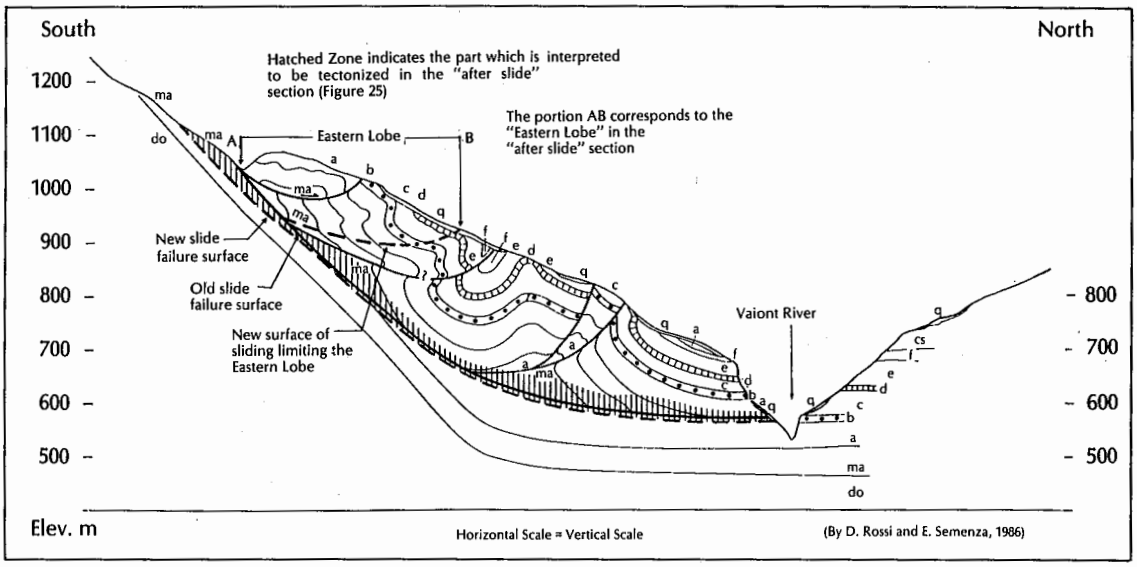


FIGURE 24. Geologic Section 10A before October 9, 1963.

Semenza have also speculated on the existence of a "tectonized" zone highlighted by the hatching along the failure surface in Figures 24 and 25. By comparing Figure 24 to Figure 25, it can be seen that there is relatively little surface deformation of the majority of the sliding mass that is not accounted for by a translational and rotational displacement, except for the deposit of the Eastern Lobe. Figure 25 also shows the near-horizontal surface of the slide mass following the 1963 slide, reflecting the very low shear strength along the base of the slide. In Figure 25, Rossi and Semenza note that a portion of the front of the pre-1963 canyon wall is missing. They have suggested that this portion fell into the gorge and was covered and spread by the slide movement. Presumably, part of this missing volume may have been removed by wave action.

An appreciation of the upstream dip of the seat of the slide cannot be obtained

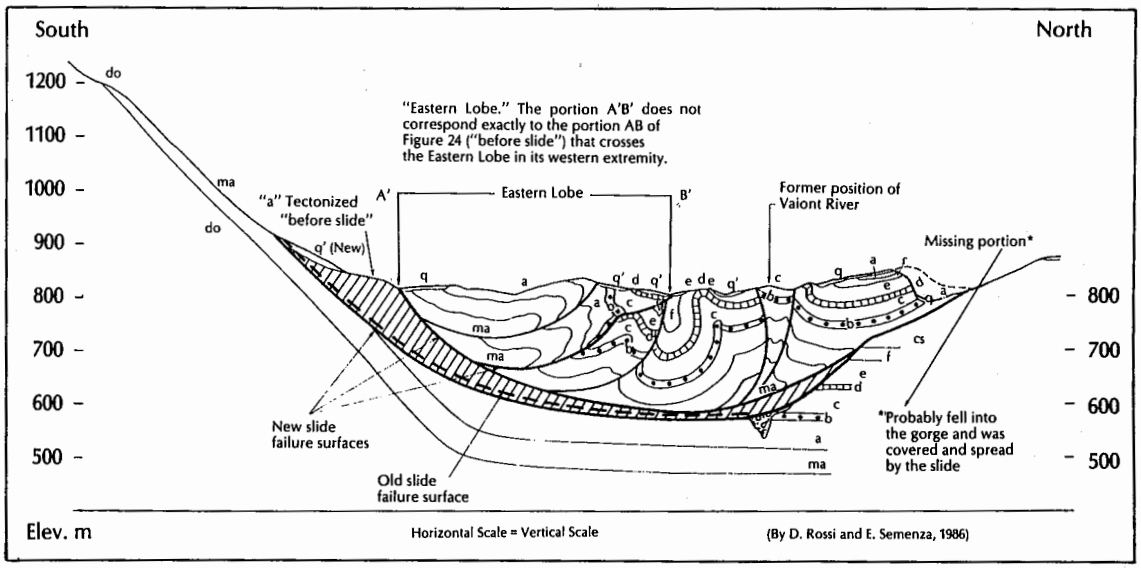


FIGURE 25. Geologic Section 10A after October 9, 1963.

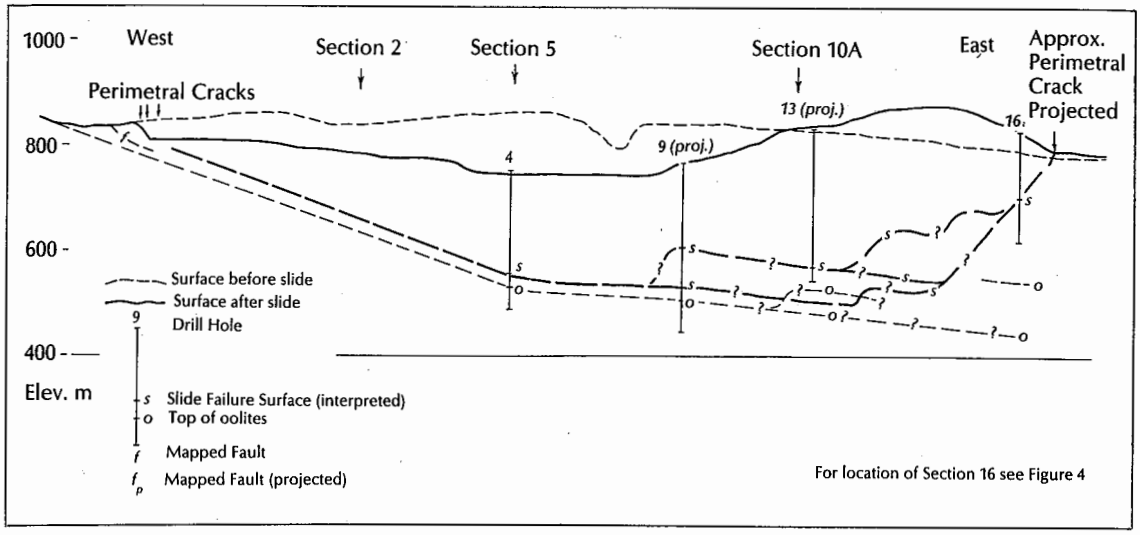


FIGURE 26. Geologic Section 16.

without an examination of an east-west section. Sections 16 and 17, shown in Figures 26 and 27, are east-west sections based on information obtained from drill-holes made after the slide. The dip of the beds along the seat of the slide is steeper (17° and 22°) on the west end near the dam and flatter (9° to 11°) in the central portion of the slide. The bedding steepens again to 30° to 40° just east of the slide (not shown in these sections). An interpretation of the stair-stepped seat of the slide on its eastern side is shown in Figures 26 and 27. The shapes of these steps are not known in detail. However, several drillholes provided local control points. A portion of one step was observed in the field. The treads of these steps will form in the weakest clay units, while the risers will form pre-existing faults and major joints.

Minor Structures

A number of folded structures were observed. One of these structures consisted of small, accordion-like, alternating synclines and anticlines with amplitudes of 2 to 15 m and wave lengths of 5 to 25 m. The axis of these folds

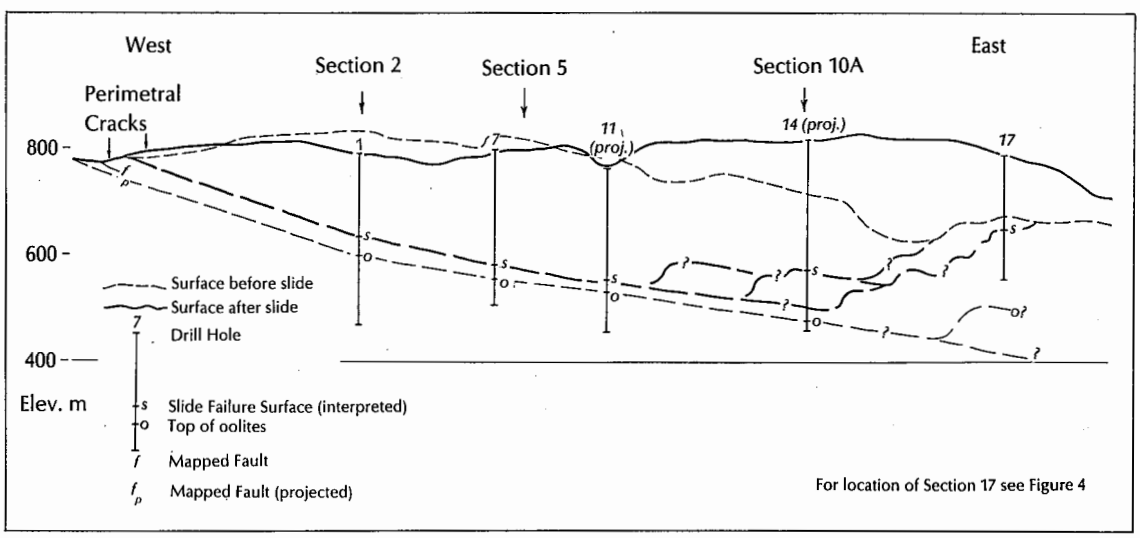


FIGURE 27. Geologic Section 17.

tends to be aligned at about 40° Az, which is within about 15° of the initial direction of movement of the slide mass. Therefore, these folds have a minimal effect on the shearing resistance of the base of the slide. However, they have had a significant effect on the distribution of the slide debris left on the in-situ bedrock surfaces. These folds underlie several of the ribs of debris that remain on the rock surfaces, especially in the western part of the slide scarp (see Figure 3). Where the folds were not aligned exactly in the direction of movement, they may have added a small geometrical component to the frictional resistance, thereby slightly increasing shearing resistance.

Perhaps the most frequently encountered structures on the exposed bedrock surfaces are the small folds and structures described as *a cascata* by Giudici and Semenza, here called cascade structures.[7] The small monoclines and cascade structures bear a dragfold relationship to the larger monocline forming the "back" of the slide. Figure 15 shows a small monocline running parallel to the strike of the beds half-way up the most westerly rock face. As the monoclinal structures develop and are subjected to continual shearing displacements, they turn into folds faulted on their bases. The stratigraphic unit forming the top of the folds continues below the cascade on the surface, but at a lower position. The deformation within the cascade structure can be complex in detail.

Small monoclines and cascade structures may serve to slightly increase the shearing resistance along the failure plane by introducing localized points of higher normal stresses that would result in some rock-to-rock contact. However, in general, the small monoclines are aligned in a stair-step fashion so that interruptions in the continuity of the clay layers are minimized with respect to slide movements to the north. The overall shear strength along a clay layer with a small monocline or cascade structure may be somewhat higher than for a smooth, continuous and uniformly dipping clay layer.

Another aspect of these folds is that they have served to preserve fragments of the clay layers that otherwise would have been eroded off of exposed bedding plane surfaces. Figure 16 is an example of a clay layer preserved in such a structure. Because of local increases in shearing resistance, the monoclinal and cascade structures also tend to collect the slide debris overlying them.

FIGURE 28. The western portion of the headscarp above location 18-6. The old headscarp is visible in the vegetated area above with a steep monoclinal fold changing to a fault at the scarp. Fragments of partly cemented breccia containing solution features remain attached to the cliff above.

The fault and associated dragfold found at the headscarp at the top of the western half of the slide are shown in Figure 28. The beds steepen appreciably close to the headscarp where they turn vertical or are faulted. Figure 28 shows that the recent headscarp is the lower portion of an older scarp whose shape is evident on the vegetated cliffs above. The fault mapped by Rossi and Semenza (shown in Figure 4) formed the eastern boundary of the slide.[26]

Fragments of partly cemented talus breccia are found along the new headscarp indicating that a peripheral crack had been opened prior to the 1960-1963 slide movements. Figure 29 shows the new scarp and the old scarp and a thick uncemented to poorly-cemented talus deposit that has filled a portion of the old peripheral crack. This crack opened up in 1960 and 1963. These old talus deposits are considered the best diagnostic evidence encountered for previous periods of movement of the Vaiont Slide in prehistoric, and perhaps during or since

FIGURE 29. A view of the western portion of the headscarp. The old vegetated headscarp is continuous with the new 1963 headscarp. Two or more types of cemented breccia have infilled old "bergschrund." Solution cavities in better cemented, probably older, breccia are visible on the right.

FIGURE 30. A float boulder of partly cemented breccia in alluvium below rock slopes whose source is believed to be breccia at the headscarp similar to that shown in Figure 24.

Roman times. A close-up view of the partly cemented talus material from the old bergschrund-like crevasse at the scarp of the slide is shown in Figure 30.

Geomorphology

Perhaps the most significant question to be addressed by geomorphic studies is whether or not there is evidence of pre-1960 slope movement. A related and very practical question is: Could the Vaiont Slide been recognized as an old slide area prior to 1961 by using conventional airphoto interpretation techniques? The latter question is particularly important to the study of other reservoir slopes, for such reviews are done principally by airphoto studies followed by field geologic mapping.[40]

In order to answer these questions, airphotos taken in 1960 and another set taken a few days after the slide of October 9, 1963, were examined. One of the 1960 airphotos is presented as Figure 31. Figure 32 shows the geomorphic features delineated from the airphoto in Figure 31. Some of the major topographical features related to the slide have been depicted in Figure 32, including depressions and scarps, streams, gullies and sinkholes. Also outlined are the dam, reservoir, roads and visible traces of trails.

Two geomorphological factors delineated in the figure are of particular interest. The first factor is a series of depressions within the slide. These depressions occur in three areas:

- the Pozza plain
- the area between the Altopiano, or high plateau above the Pozza, and the cliff that traces the location of the dragfold, monocline or fault at what will develop into the headscarp of the western side of the 1963 slide
- the area of large scarps, one below the other, and small depressions on the eastern half of the slide

The eastern and western limits of the 1963 slide are defined in the 1960 airphotos by an abrupt change in morphology or by airphoto lineaments. The depressions in these three areas appear to be primarily the result of previous slide movements that occurred several thousand years ago. However, the depressions were no doubt enlarged by solution of the carbonate rocks present. Kiersch mapped a number of these depressions within the slide boundary and described them as sinkholes.[10] The time between the occurrence of the original landslide and the 1960 airphotos was sufficient for erosion to subdue the original landslide topography so that the evidence is not particularly obvious. However, it seems likely that, after detailed

study, an experienced airphoto interpreter would recognize the area as a possible or probable landslide. Certainly, on-the-ground field investigations would be required to confirm such an interpretation. The appearance of the slopes above the slide, and west and northwest of the slide, suggest that they have been denuded by previous slides.

The second geomorphical feature of particular interest, and one of the most surprising aspects of the airphoto study, was the substantial area of pronounced karstic topography in a basin above the slide and to the west of the peak of Mt. Toc. Other small incipient sinkholes are present in the surface of the Dogger beyond the western and southern limits of the slide. These apparent kettles or sinkholes, which sometimes form small elongated doline-like depressions, are mapped in Figure 32.

Hydrogeology

The principal reason for studying the groundwater conditions within a slide is to determine the distribution of the water pressures acting along the sliding surfaces. When the average rainfall of an area is in the range of 1,200 to 2,300 mm/year and the terrain is mountainous, there is the potential for significant fluctuations in groundwater pressures and levels to occur. Detailed precipitation records for the village of Erto from 1960 to 1964 were supplied by ENEL.

The groundwater data available for the Vaiont Slide area are sparse and, unfortunately, questionable. The data consist of water levels measured in three drillholes (P1, P2 and P3) from the summer of 1961 until October 1963. The locations of these drillholes are shown in Figure 4. Water level measurements were made inside pipes placed in open drillholes. The annulus between the pipe and the rock was not sealed so that water pressures at different elevations in the rock would be expected to be connected (see Figure 33b). As a result, the water levels recorded inside the casing could reflect some average value of the different water pressures and hydraulic conductivities of the units encountered. However, if a natural seal developed on the outside of the pipe (for example, by a soft clayey layer squeezing around the pipe), then the water level inside the pipe would reflect average hydraulic pressure conditions in the formations below the seal as shown in Figure 33c. Such a seal could conceivably provide water pressure readings in the vicinity of the base of the slide as precise as if a fully-sealed standpipe piezometer, such as that shown in Figure 33a, had been installed. With continued but small displacements of the slide, the seal around the pipe could be eroded or the pipe could bend or be pulled apart and start to leak as shown in Figures 33e and 33f.

From early November 1961 (when P2 was first read) until late January 1962, the water level in P2 was 25 to 90 m above the reservoir levels. During this period the slide moved 5 to 10 cm. From February to July 1962, piezometer P2 showed levels lower than previously indicated, but still 2 to 10 m higher than piezometers P1 and P3. During this interval the slide had moved from 20 to 25 cm (since P2 was installed). After July 1962, the water levels recorded in P2 were generally within 1 to 2 m of those recorded in P1 and P3. Thus, the total displacement of the slide since P2 was installed was about 30 cm by July 1962. This much displacement was probably sufficient to pull out, rupture or pinch the end of the pipe in the manner suggested in Figures 33e and 33f.

One piezometer (P2) out of the three was apparently partially sealed and, for a period of about two months, gave more representative water pressures of conditions near the surface of sliding than the others. The other piezometers, P1 and P3, probably gave measurements that were representative of the groundwater table in the highly fractured rock mass above the basal clay-rich zone. The groundwater pressures in the bulk of the rock debris appear to have varied directly with reservoir levels, maintaining a slight (3 to 10 m) increase above reservoir levels. No groundwater data appears to have been obtained, or recorded, from the holes drilled after the 1963 slide.

The scarcity of groundwater data makes it important to develop a reliable concept of the basic hydrogeological conditions at the slide in order to make reasonable assump-

FIGURE 31. Airphoto of the Vaiont Dam and Reservoir area taken in 1960.

tions of water pressures for stability analyses. A knowledge of groundwater flow systems can be used to predict the typical pressure distributions to be expected.

Figure 34 on page 97 shows the general groundwater flow system that might be expected on a section through Mt. Toc, assuming a relatively homogeneous and isotropic distribution of hydraulic conductivities within the mountain. If on the northern slopes of Mt. Toc there was a tendency for higher conductivities along the bedding than across the bedding, there would be a corresponding tendency for the higher fluid potentials originating from infiltration in the area of the karstic topography on the upper slopes of the mountain to be transmitted to the Vaiont Slide region with minimal head losses. At the base of the Vaiont Slide mass, high fluid potentials would be held beneath the clay layers, whereas in the highly fractured rock above the more continuous clay

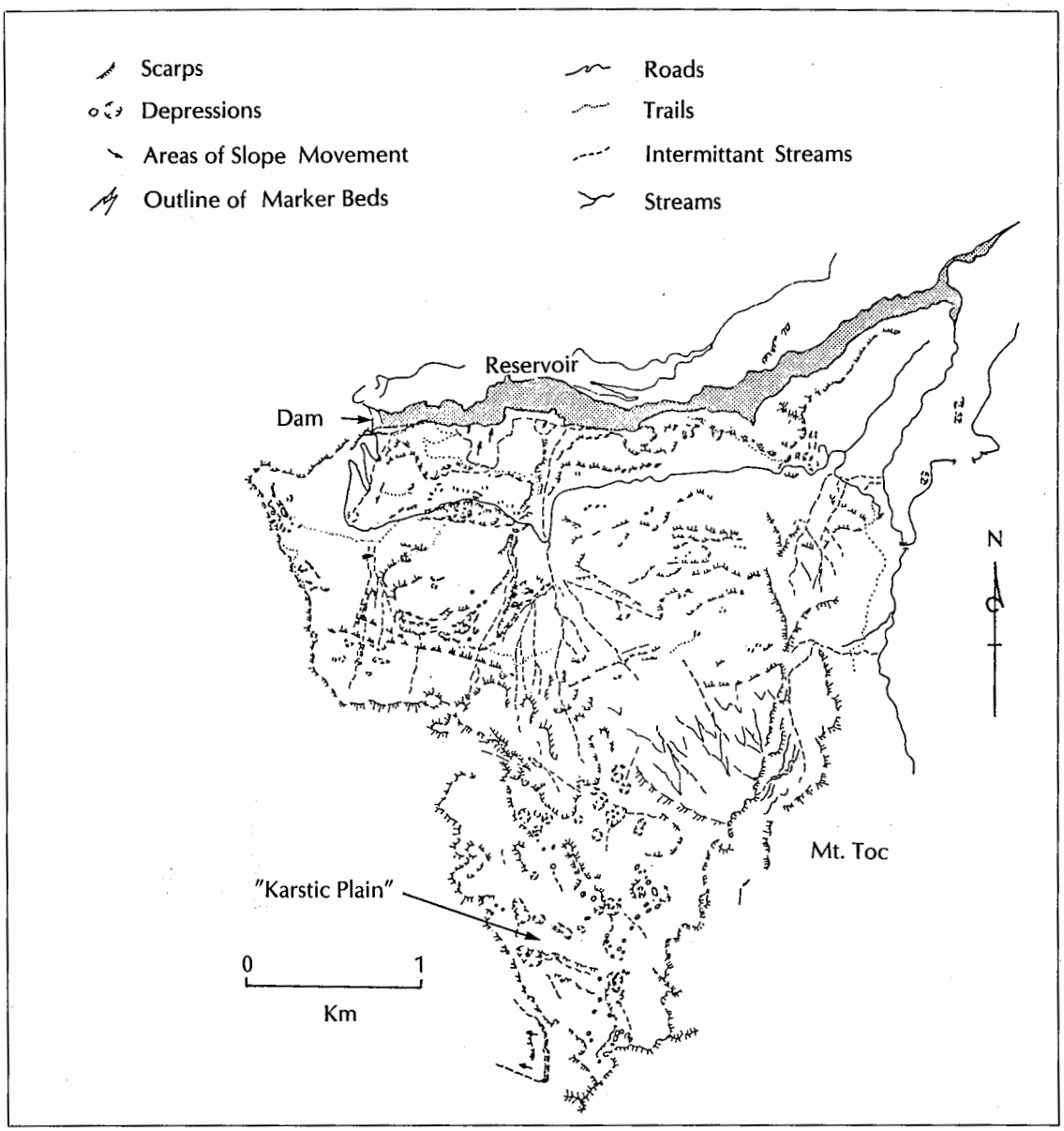


FIGURE 32. Geomorphic features of the Vaiont Slide area delineated from the 1960 airphoto (same scale as Figure 31).

layers, the fluid potential would be much lower, reflecting the fluid potential base level in the valley. The base level for the local groundwater pressures in the portion of the slide above a clay interbed is likely to be either the elevation of the intersection of the top of the clay with the valley wall or the reservoir level, whichever is highest. Beneath and within the zone of clay interbeds, the water pressures should vary with changes in the groundwater conditions (or levels) at the top of the mountain and with changes in the outlet pressure conditions in the valley at the base of the mountain. Therefore, the water pressures below the clay layers should directly reflect changes in infiltration rates because of rainfall or snowmelt above the slide and changes in reservoir levels. Kiersch was the first to comment on the importance of infiltration on slide mass stability.[10]

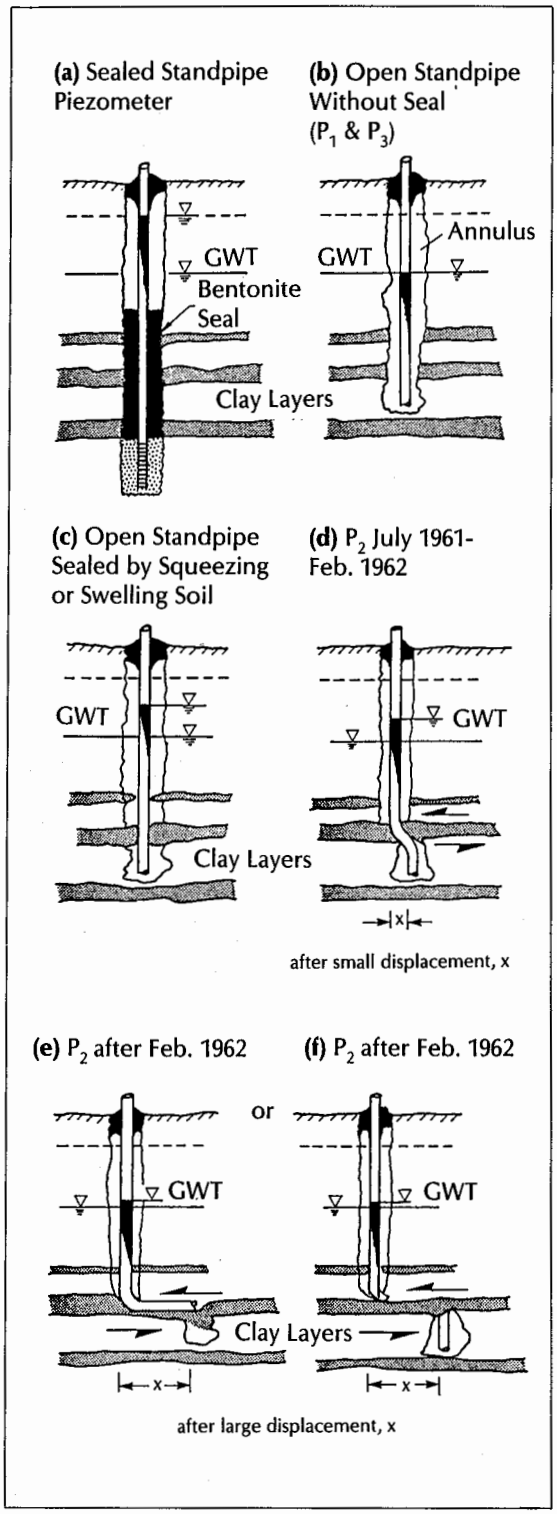


FIGURE 33. Sketches showing a possible explanation for the water levels recorded in P2.

Initial fluid pressures recorded in P2, which were about 90 m above reservoir level, occurred during a period of moderately low precipitation. These fluid pressures increased approximately 20 m during a period when a 20 m rise occurred in the reservoir level. This increase implies that the relatively closed groundwater flow system described above was present near the base of the slide. In such a system, changes in the outlet pressure could have an effect at appreciable distances away from the reservoir. Thus, the hydrogeological conditions present appear to provide the opportunity for large groundwater pressure fluctuations to occur around the base of Mt. Toc. The very limited piezometer measurements available support this view.

During a conversation with the authors, E. Semenza recalled that he had observed springs and moist patches in two areas where, in 1959 and 1960, he and Giudici had mapped the exposed shear zone that forms the outcrop of basal failure plane in the Vaiont Canyon. The outcrop of the remainder of this plane was beneath a rock talus formed by the raveling slopes above. Semenza's description of these groundwater discharge areas is consistent with the hydrogeological picture noted above.

Solution cavities were observed at four locations on the exposed scarp in the rocks immediately below the failure surface. The cavities ranged in size from 0.5 to 50 cm in diameter. The solution cavities would suggest that hydraulic connections existed beneath portions of the failure surface. Undoubtedly, other solution cavities could have been located if time was spent investigating these features. Solution cavities are most likely to be associated with small faults and folds in the bedding, and with beds that are more susceptible to solution than others.

During field visits to the Vaiont Slide, the authors observed that during moderate to heavy rainfalls no water flowed from the Massalezza Ditch onto the slide scarp, although many of the drainage paths down the scarp become torrents. This lack of flow in the Massalezza is believed to be indicative of the very high infiltration of precipitation into the karstic bedrock on the slopes above the slide

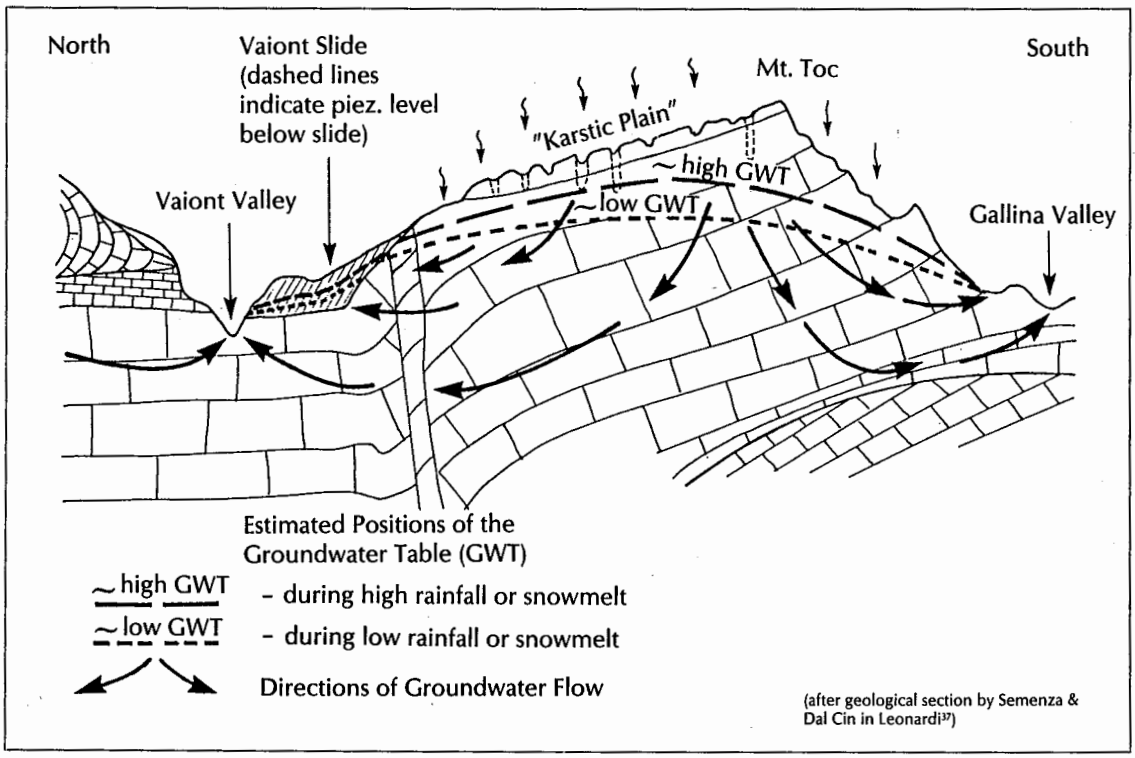


FIGURE 34. A schematic section through the Vaiont Slide showing the estimated regional groundwater flow system.

scarp. Presumably, water would flow in the upper Massalezza Ditch after heavy and prolonged rainfalls or snowmelts. But when this happens, the groundwater pressures in the underlying rocks would have to be much higher than when the Massalezza runs dry.

One of the main purposes of the adits placed in 1960-61 on either side of the Massalezza Ditch (see Figure 4) was to investigate the possibility of draining the slide.[20] One of these adits was encountered by the authors on the east side of the Massalezza. The top of the portal of this adit had approximately 1 m of cover below the exposed failure surface. Since these adits were so close to the Massalezza, which generally was dry, it is not surprising that very little water was encountered in them. The adits were located too high in the slide and too close to the ground surface to encounter the high water pressures that were undoubtedly present at greater depths and in more representative portions of the slide. This placement was unfortunate, since conclusions drawn from the lack of water encountered in adits were reportedly responsible for the 1961 decision that it was not practical to stabilize the slide by drainage.[41]

In 1979, E. Semenza described for the authors the sheared clay-rich zones (ultramylonites) that were exposed in the adits of the western side of the Massalezza and along the adjacent stream bed and noted earlier by Semenza.[12] Little attention has been given to these clay-rich zones in the literature. They, no doubt, were outcrops and subsurface exposures of previous or potential slide planes.

Lo *et al.*, and others, speculated on the probable existence along the base of the slide of artesian pressures.[42] Generally, these writers assumed that the pressure distribution along the failure surface followed a straight line from the reservoir to the top of the slide. Such an assumption for the distribution of fluid pressures results in piezometric levels that are much higher than the initial readings of piezometer P2 indicated. The water pres-

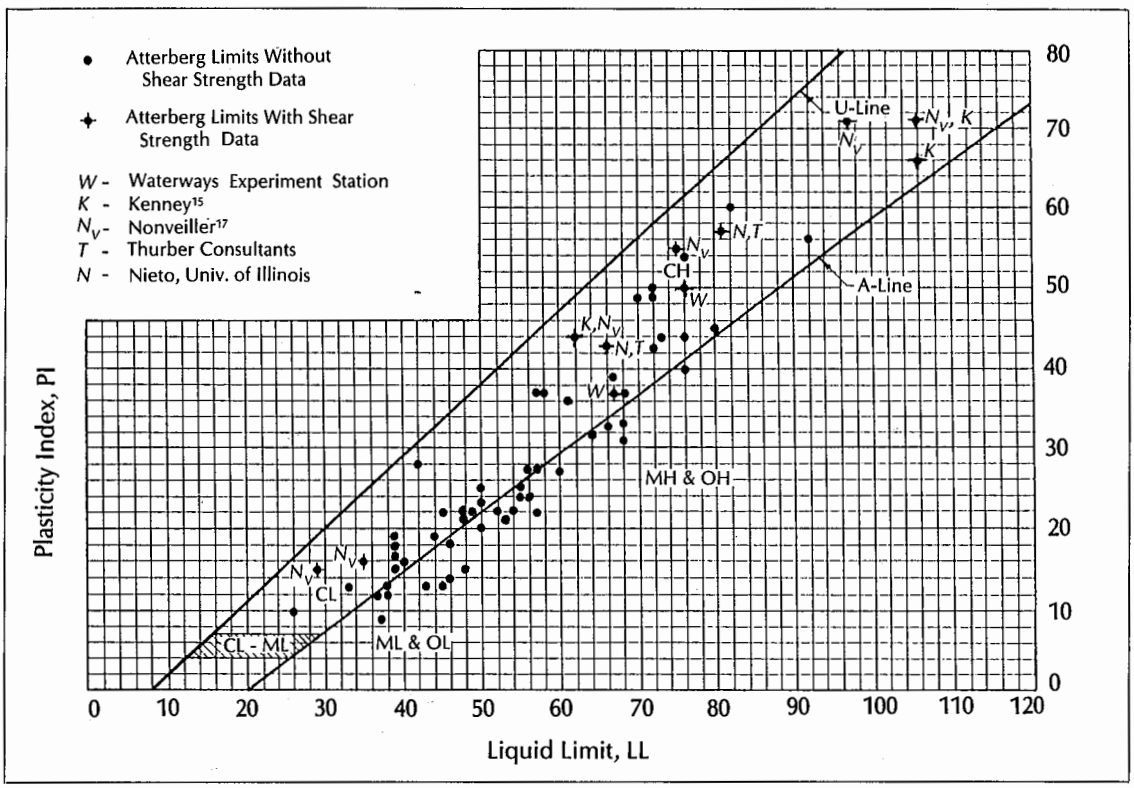


FIGURE 35. Plasticity chart for the clay samples from the slide site.

sure distributions used in the analyses in this study were made to agree with the initial P2 record for low rainfall conditions and were increased for high rainfall conditions.

Physical Properties of the Clays

The properties of the clayey materials found along the failure surface of the Vaiont Slide were tested by soil laboratories in several different countries during the course of this study.[43] The tests were conducted over a five-year period from 1976 to 1981. The initial tests in 1976 and 1977 were made for work on the Downie Slide undertaken by the authors for the British Columbia Hydro and Power Authority. The results of all these tests are presented in this section together with the results of tests on the Vaiont clay layers published by others. The tests performed include grain-size analyses, Atterberg limits, direct shear strength tests and clay mineral analyses.

A grain-size analysis completed on one of the clay samples indicated 51 percent clay, 36 percent silt, 7 percent sand and 6 percent gravel. Samples of the Vaiont clays were also examined by Kenney.[15] He reported similar results, with 52 to 70 percent of his samples less than 2 microns in size.

Atterberg limits are more directly related to the strength properties of the soil than are the grain-size analyses. Therefore, most samples were tested for their liquid and plastic limits. These results are presented in Table 2. Figure 35 is a plasticity chart that shows the Atterberg limits of the clay samples obtained.

The results are well distributed over a large range of liquid and plastic limits. However, there appears to be a tendency for the samples to fall within two general groups. One group plots nearly on the A-line, and the soils are classified as CL, ML and MH. Thus, these soils are inorganic clay of low plasticity and inorganic clayey silts of low to high plasticity. The liquid limits of this group vary from 33 to 60 and the plasticity indices vary from 9 to 27. The other group falls within the area of the plasticity chart distinctly above

the A-line. These soils are classified as clays of high plasticity (CH soils). In this second group, the soils had liquid limits that varied from 57 to 91 and plasticity indices that varied from 30 to 61.

Results of tests by Kenney and Nonveiller for clay samples from the Vaiont Slide are also shown on Figure 35.[15,17] The liquid limits (29 to 116) and plasticity indices (15 to 71) of some of these samples exceed those measured in this study, but are consistent with the character of the clays noted in the field observations.

Shear strength test results are summarized on Tables 3, 4 and 5. Test results given in Tables 3 and 5 were conducted by reversing the direction of movement in a direct shear box, whereas the tests in Table 4 were made with a direct shear box that permitted approximately 5 cm of travel in one direction. All shear strength tests were made on samples of the remolded clayey soils.

Two series of tests were performed on one sample in Table 3. These tests were made at stress levels of 103 to 6,200 kPa (15 to 900 psi) and the surfaces of sliding were pre-cut. The first series of tests were conducted on that portion of the sample (83 to 87 percent by weight) passing the No. 10 sieve size. The second series of tests were conducted on the material that passed the No. 4 sieve. The thickness of the samples was 7 to 11 mm, and the rate of shearing varied from 1 to 0.003 mm per minute. The area of samples was 25.8 cm^2.

The results indicate a range of values from 5.9° to 16.4° for the drained residual angle of friction, with higher values generally being obtained from tests run at lower stress levels. If results from tests made at stress levels under 350 kPa (50 psi) are ignored, the results for samples 522-5 and 522-5A range from ϕ_r = 9.6° to 11.4° for "whole" samples and 7.4° to 8.5° for the fine-grained portion of the samples. The results for sample 11-9 for stress levels of 350 to 1,030 kPa (50 to 150 psi) range from ϕ_r = 5.9° to 9.6°.

The results of the clay mineral analyses are presented in Table 6 on page 102 together with the results of Kenney.[15] The clay mineral analyses indicate that some 50 to 80 percent of the whole samples are clay minerals predominantly of the type generally known in soil mechanics as calcium montmorillonites. However, in detail, the clays are composed of 25 to 75 percent of a mixed layer vermiculite/smectite composition with the remainder of hydrous mica illite to smectite composition containing something on the order of 60 percent smectite. An illite/corrensite composition is reported in one set of analyses. Such clay materials have an expanding lattice, are associated with low shear strengths and exhibit swelling properties when stresses are reduced and water is present. The residual angles of shearing resistance obtained from these samples compare favorably with the 8° to 10° reported by Olson for calcium montmorillonite.[44] Olson's tests were run with stress levels of 350 to 500 kPa (50 to 75 psi).

Correlations Between Reservoir Level, Precipitation & Rate of Movement

A chronological list of the events leading up to and following the Vaiont Slide is given on pages 106 - 111. Natural events, construction activities and the activities of engineers and geologists investigating the slide area have been included as an aid in understanding many of the technical aspects of the slide.

Comparisons between precipitation in 10-day intervals with reservoir level, rate of slide movement and water level in the piezometers from 1960 through 1963 are presented in Figure 36 on page 104. There was a small slide in March 1960 at the toe of the east end of the overall slide (shown in Figure 4). The time of this slide, which occurred before displacement measurements were made, is shown in Figure 36. The March 1960 slide occurred without a noticeably high 10-day incremental rainfall, although there were substantial 3-day rainfalls. This early slide also could have been associated with a period of snowmelt and probably was strongly influenced by the rising reservoir.

The first major slope movement that was monitored occurred in October 1960 during the first filling when the reservoir level had reached an elevation of about 650 m. By late October 1960, the displacements were sufficient to result in a series of cracks that

TABLE 3

Summary of Direct Shear Test Results on Remolded Vaiont Clays — Group 1

Sample No.	Water Content of the "as received" Soil (%)	Atterberg Limits LL (%)	PL (%)	PI (%)	Shear Test & Specimen Details: Test Conducted on	Type of Test
Vaiont Sample 522-5A	26.2	66.2	22.5	43.7	Soil after removing all rock & coarse sand retained above sieve No. 10. About 13-17% by weight of total sample was removed which constituted the rock fragments & coarse sand.	Multistage direct shear test along a precut plane.
Reconstituted Vaiont Sample 522-5A	26.2	81.0	23.8	57.2	*Reconstituted* sample after adding back the coarse sand fraction between sieve Nos. 4 & 10 to the above sample. However, rock fragments were not added.	Multistage direct shear test along a precut plane.

Note: Results shown in this table were from tests performed by Thurber Consultants Ltd., Edmonton, Canada. Grain size distribution for the original sample was: Gravel 6%; Sand 7%; Silt 36%; and Clay 51%.

TABLE 4

Summary of Direct Shear Test Results on Remolded Vaiont Clay — Group 2

Sample 522-5

Vaiont Test No.	Test Type*	Deform. Rate (mm/min)	Initial Normal Stress (kPa)	Peak Shear Resist.** (kPa)	Displ. at Minimum Resistance (cm)	Normal Stress at Min. Resist. (kPa)
1W	1	0.0635	576	161	2.89	712
2W	1	0.0635	576	143	2.34	681
		0.00635			3.48	747
3W	1	0.0635	421	93.8	1.55	467
4W	1	0.0635	576	147	1.98	665
					2.41	685
5W	1	0.0635	576	154	2.34	678
6W	1	0.0635	576	150	2.97	713
7W	1	0.0635	288	89.6	2.54	346
					5.41	246
8W	1	0.0635	288	93.1	5.21	438
9W	1	0.0635	157	53	5.33	242
1F	2	0.0635	576	131	2.95	713
2F	2	0.0635	288	71	5.21	350
3F	2	0.0635	576	152	3.18	730
					3.68	381
					4.42	221
					5.51	572

Note: Results shown in this table were from tests performed by the Engineering Geology Lab, Dept. of Geology, Univ. of Illinois-Urbana. *Test Types: 1 = Whole Sample; 2 = Fraction passing #140 mesh; 0.15 cm sample between two 5 x 15 cm slabs of Berea sandstone unless otherwise indicated. **Displacement at peak resistance assumed to be zero.

Remolded Water Content (%)	Normal Stress σ_n (kPa)	Post-Shear Water Contents: Shear Plane	Post-Shear Water Contents: Away From Shear Plane	Residual Shear Strength τ_{res} (kPa)	Effectual Residual Strength Parameters: tan ϕ_r τ_{res}/σ_n	Effectual Residual Strength Parameters: ϕ_r
27.0	6205	25.9	25.3	810	0.131	7.44°
	1724			252	0.146	8.3°
	345			55	0.16	9.1°
30.0	6205			1047	0.169	9.6°
	103			30.3	0.293	16.4°

Minimum Shear Resistance (kPa)	Tan ϕ	ϕ deg.	Remarks
144	0.203	11.4	Added only enough water to work sample into a 0.15 cm layer.
126	0.185	10.5	Added additional water; allowed sample to soak for two days.
134	0.181	10.2	Ran sample for about 0.25 cm to test effect of deform. rate of ϕ_M. Ten-fold decrease in deform. rate resulted in 2% drop in ϕ.
70.3	0.150	8.5?	
114	0.172	9.8	Resistance increased slightly after passing through min. value (see next line).
121	0.177	10.0	
114	0.167	9.5	
122	0.171	9.7	
68	0.195	11.0	Sample was unloaded after reaching residual. Thin sample. Normal load = 1223 N.
47	0.193	10.9	
83	0.189	10.7	
54	0.223	12.5	
99	0.139	7.8	
48	0.135	7.7	
109	0.150	8.5	Sample was unloaded after reaching residual.
56	0.146	8.3	Normal load = 2237 N.
33	0.149	8.5	Normal load = 1214 N.
84	0.147	8.4	Normal load = 2842 N.

TABLE 5

Summary of Direct Shear Test Results on Remolded Vaiont Clays — Group 3

Sample 11-9

Specimen No.	Atterberg Limits LL	PL	PI	Initial Water Content %	Dry Density N/m³	Initial Void Ratio	Saturation %	Final Water Content %
1	76	26	50	35.4	13480	0.998	97.5	30.2
2	67	30	37	30.3	13866	0.944	88.3	27.7

Note: Results shown in this table were from tests performed by the Waterways Experiment Station, Vicksburg, MS. Shear plane precut & sample description: plastic clay (CH), gray.

TABLE 6

Summary of Clay Mineral Analyses on Vaiont Samples

Laboratory	Sample No.	Results
WES (A.D. Buck) (See Table 5)	1. Clay, 11-9	smectite-major component (50%) calcite-minor component quartz-minor component kaolinite-minor component
	2. Limestone (fine-grained greenish-gray)	calcite-major component quartz-minor clay & mica-minor "randomly mixed-layer"-smectite & vermiculite-minor
Dept. of Geology Univ. of Illinois (Dr. Eberl) (See Table 4)	1. Whole rock (*i.e.*, clay sample) 522-5	calcite-major component corrensite illite/smectite quartz (decreasing order of abundance)
	2. Less than 2-micron fraction 522-5	corrensite* (vermiculite/smectite type) illite/smectite* (-60% smectite layers) calcite quartz-small amount (* = present in approx. equal proportions)
Alberta Research Council (Thurber) (See Table 3)	5A. 522-5A	illite hydrous mica mixed layer clay minerals containing montmorillonite

Kenney[15]

Massive Minerals (% Dry Weight)

	Qtz	Feldsp	Calcite	Others	Total	ϕ_r
Vaiont I	5	5	30	10	50	10.25°
Vaiont II	10	—	7	5	22	9.0°
Vaiont III	5	—	40	—	45	15.75°

Clay Minerals (% Dry Weight)

	Kaolin	Chlorite		Mica (Hydrous mica Illite)	Mixed Layers with Mont-morillonite		Mont-morillonite
Vaiont I	—	—			——	50	——
Vaiont II	—	——	5	——	——	75	——
Vaiont III	—	——	30	——	——	25	——

Estimated Specific Gravity	Type of Test	Normal Stress kPa	Residual Shear Stress kPa	Deform. Rate mm/min	Displ. at Estimated Shear cm	tan ϕ	ϕ_r
2.75	Shear test along a precut surface	171	23.0	0.0089	9.52	0.134	7.6°
		345	37.9		19.1	0.110	6.27°
		689	86.8		24.1	0.126	7.18°
2.75		517	53.8	0.0089	4.6	0.104	5.9°
		1033	175		16.3	0.169	9.6°

essentially outlined the perimeter of the entire slide as it subsequently developed in 1963. This perimeter crack is shown in Figure 4 together with the outline of the October 9, 1963, slide. Figure 36 demonstrates that the development of the perimeter crack coincided with the maximum 10-day precipitation for the year. Also, the onset of significant movement coincided with the start of a period of unusually heavy and prolonged precipitation that followed an exceptionally wet July and August. The slide continued to move after the perimeter crack opened, reaching a maximum rate of 3 -4 cm/day at the end of October 1960.

On November 4, 1960, a major slide occurred along the toe of the future Vaiont Slide and some 700,000 m^3 of material slid into the reservoir. The outline of this slide is shown in Figures 4 and 20. The reservoir level was lowered immediately after the November 4 slide from a maximum level of 650 m and reached el. 600 m by early January 1961. Thereafter, slide movements decreased rapidly to less than 0.1 cm/day. The slide essentially stopped moving when the reservoir level was below el. 600 m and when the precipitation was low.

At the end of the first drawdown, the average total displacement on the western half of the slide area was 100 cm, and the total movement east of the Massalezza Ditch was less than 20 cm. Figure 36 shows that the decline in the rate of movement of the slide from November 1 to 4, 1960, corresponded to the end of an abnormally high rainfall. At this point the reservoir level was still rising.

The reservoir was held between el. 585 and 600 m from early January 1961 until early October 1961. During this period, a bypass tunnel was driven into the right bank of the valley opposite the 1963 slide area. This period was one of moderately low precipitation except for one wet 10-day stretch in May 1961. The rate of movement of the slide during this period was negligible. Piezometers P1, P2 and P3 were installed during this period and water level readings commenced as shown on Figure 36.

The second filling of the reservoir began in October 1961, and near the end of January 1962 the reservoir elevation was again at 650 m. As Figure 36 indicates, the rate of movement corresponding to the second filling to el. 650 m was negligible and the velocity was less than 0.1 cm/day. This behavior was in sharp contrast to the 3.5 cm/day velocity observed when the reservoir was just below el. 650 m during the first filling. Even as the reservoir approached el. 700 m at the beginning of November 1962, the velocity was only about 0.2 to 0.3 cm/day. The rate of movement increased abruptly to about 1.2 cm/day at the end of November 1962, although the reservoir remained nearly constant at the 700 m elevation. This increased movement followed a period of record precipitation for the four-year period shown on Figure 36. The reservoir was lowered to 650 m by the end of March

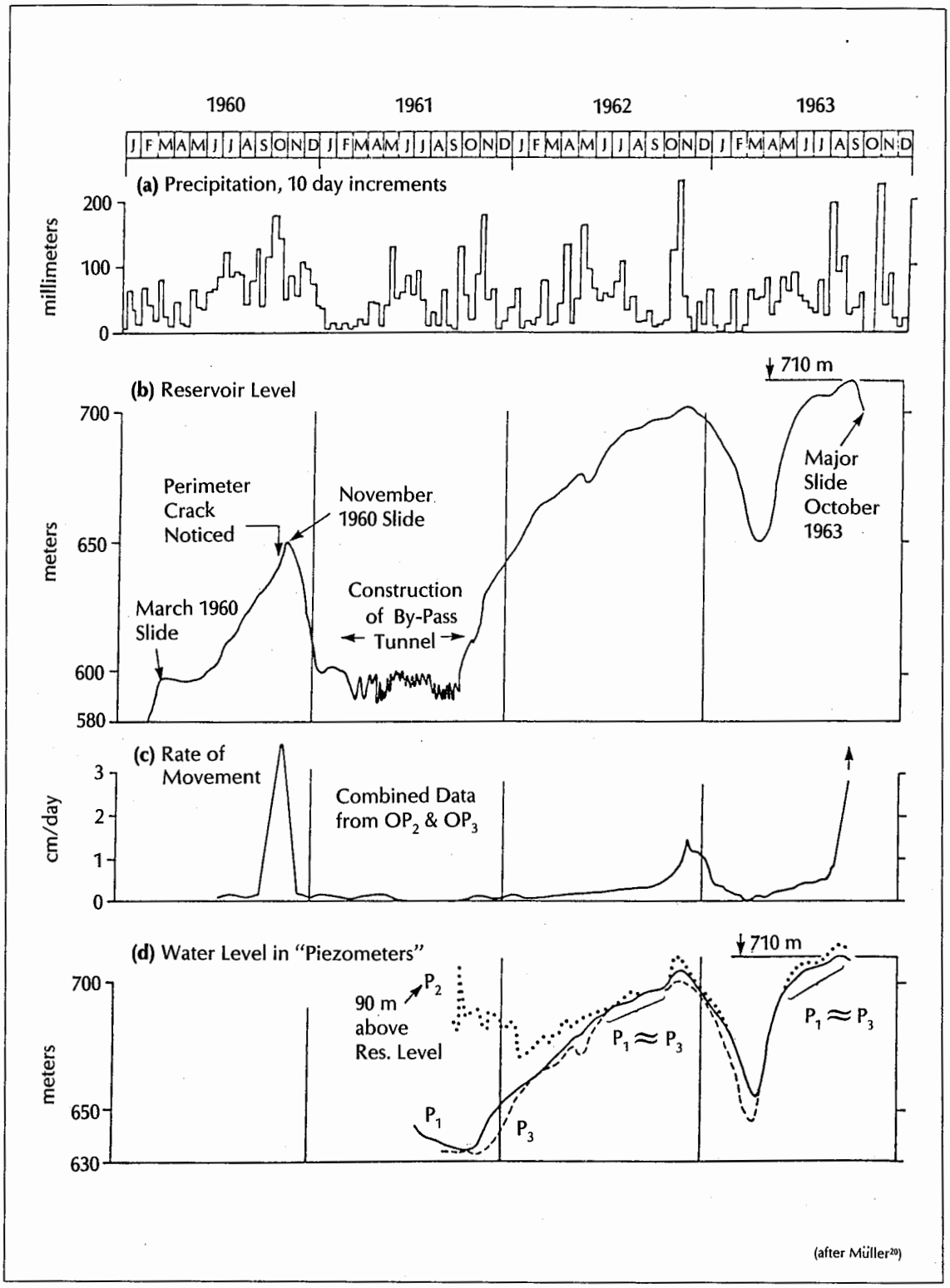


FIGURE 36. A comparison of the water levels, slide movements and precipitation from 1960 to 1963.

1963 and the movement stopped. But the movements that occurred during the second filling and drawdown to el. 650 m amounted to 130 cm. These movements were in addition to the 100 cm of movement that occurred due to the first filling.

The third filling of the reservoir began in April 1963, and the reservoir reached approximately el. 695 m by early June 1963. At this time the slide velocity was about 0.3 cm/day (see Figure 36). The reservoir reached 705 m in the middle of July 1963, and the rate of movement increased to about 0.4 to 0.5 cm/day.

In mid-August, the reservoir started to rise from el. 705 m and reached 710 m in early September. There was an immediate increase in the rate of slope of movement from 0.5 to 1.0 cm/day. This rate continued to increase throughout September, reaching 2 to 4 cm/day in the first days of October. In early October, lowering of the reservoir began. The elevation of the reservoir had dropped to about 700 m by October 9, 1963, when the major slide occurred. According to the report of the Bozzi Commission, the velocity of the slide by that day was about 20 cm/day.[45] Figure 36 shows that the final acceleration of movement began in late August 1963 and coincided with a period of near-record precipitation for the four-year period.

If other factors governing stability remain constant, then it is reasonable to assume that similar rates of movement should be observed for similar reservoir levels. The empirical observations at Vaiont, which showed that the movement of the slide in October 1960 was 3.5 cm/day when the elevation of the reservoir was at 650 m, seemed at variance with the observation of a negligible slide velocity in January 1962 when the reservoir was also raised to el. 650 m. Moreover, the rate of movement of 1.2 cm/day, which accompanied the reservoir elevation of 702 m in November 1962, was below the rate of 3.5 cm/day observed for the 650 m reservoir elevation in October 1960. Having such data available, Müller stated:[20]

> "The experiences gathered during the second period of storage seemed also to confirm the assumption, developed in the meantime, according to which it was considered possible to control the velocity of the slide by the effect of the water on the sliding mass itself. The observation that the movements generally had a higher velocity only if a new portion was wetted for the first time, whereas they remained always smaller than the previous one if a layer once wetted was flooded the second time, led authorities and technicians to the conviction that a gradual stabilization of the moving mass would be brought about by raising the water level in individual steps. It was assumed that the mass would eventually reach a certain equilibrium, or, at least would keep moving so slowly that no serious problems would occur" (p. 178).

The erroneous assumption that led to the conclusions quoted above was that all other factors were remaining constant and the reservoir level was the main variable controlling the stability of the slide. In fact, rainfall was significant and was not remaining constant. Figure 36 reveals that there were periods of high precipitation preceding all the major slide movements in October 1960, November 1962 and October 1963. In addition, when there are different movement rates for similar reservoir levels, the higher movement rate correlates with a higher precipitation rate. For example, the second time the reservoir reached el. 650 m in January 1962, there was negligible movement because of the low precipitation at that time and in the preceding months. Another example can be seen by comparing the rates of movement for June 1963 when the reservoir was at el. 700 m with those for November 1962 at the same reservoir level. The rate of movement was accelerating in November 1962 following a period of record precipitation, whereas in June 1963, with near normal precipitation, the rate of movement was nearly constant and at half the rate observed in November of 1962. Thus, from an evaluation of the records shown in Figure 36, rainfall appears as important as the reservoir level in determining the rate of movement of the slide.

The history of the three slide movements

Chronology of Significant Events

1928 Prof. Giorgio Dal Piaz examined the stability of the future banks of the reservoir. At that time no question was raised about the area of the 1963 slide.[20] The Bozzi Commission indicated that while Dal Piaz described a general phenomena of deep fissures near the Casso bridge, nevertheless the reservoir conditions were no worse than those met on the great majority of the mountain basins throughout the Venetian area.[45]

1956-57 Excavation at the dam.

July 1957 Construction of the dam began.

1957 Müller was consulted on problems of the stability of the rock abutments of the dam and on how the stability of the future reservoir banks should be determined. On the basis of a short inspection of the rock fabrics, Müller thought it possible that the reservoir "would cause slides, some of which might be perhaps as much as 1 million m^3 in some parts of the future banks" (p. 156).[20]

1958 Dal Piaz reexamined the stability of the left valley slopes between the Pineda and the dam in connection with the construction of the new road along this bank. He concluded that the rock was fractured, but it was in place and showed no signs of an earlier movement with the exception of a small strip 500 m east of the Pozza where the rock was covered with moraine-like materials. Dal Piaz concluded that only local detachments of such materials could be expected; however, these would not be of a serious magnitude (p. 157).[20] The Bozzi Commission noted that Dal Piaz included a discussion of fissures in the area of the Pozza in his report.[45]

Spring 1959 Carlo Semenza, designer of the dam, invited Müller and E. Semenza, geologist, to inspect the banks of the future storage reservoir.[12] Semenza noted that following this visit, Müller, in his report No. 6 of 1959, outlined a general investigation program to assess the stability of the Vaiont banks.

Summer 1959 F. Giudici and E. Semenza conducted a geological survey of the banks of this proposed reservoir.[7] Field observations made during this survey led to the first doubts regarding the stability of the left bank.[12] (Much of the earlier concern had been for the Erto area). An uncemented mylonitic zone, extending some 1.5 km along the left wall of the Vaiont Canyon, was identified during this survey. Question marks on the geologic sections (see Figure 7a) indicated the authors' uncertainty about the upslope extent of a possible slide mass associated with this fault.

Rock masses with disturbed bedding were found lying on gravel and sand deposits on the righthand side of the Vaiont Valley. On the basis of these facts, it was then hypothesized that the area from the Pozza down to the Vaiont River represented the mass of an old prehistoric slide that moved down Mt. Toc in a northeast direction.

Oct. - Nov. 1959 Caloi started a

can be explained by considering the combined effects of precipitation and reservoir elevation. It is not necessary to consider another mechanism, such as "creep" or "thixotrophy," because the behavior only appears to be anomalous when the movements are correlated with reservoir levels without considering precipitation.[3,4]

As heavy rainfall or snowmelt penetrated into the slopes of Mt. Toc, uplift pressures could be generated on the failure surface corresponding to piezometric levels much higher than the reservoir level. At low reservoir levels, very heavy rainfalls would be required to develop uplift pressures large enough to cause slide mass instability. As the reservoir

seismic survey of the Massalezza area along a profile that lay roughly parallel to the Massalezza Ditch and extended from the Vaiont Gorge to el. 850 m. The total length of the two traverses completed came to about 1 km. Caloi interpreted the results to be proof that the left valley wall consisted of "extraordinarily firm in-situ rock — covered with only 10 to 20 m of a loose slide material. Thus the hypothesis of an ancient very deep slide of the in-situ rock became improbable" (p. 159).[20]

In a letter to his father, Carlo, in April 1960, E. Semenza differed with Caloi's conclusions.[12] Semenza indicated that, in his opinion, the left side of the valley was a large rock mass that had slid in the past in a northeast direction. This opinion was discussed by Giudici and Semenza in their report submitted in June 1960 and was also noted by Müller.[20]

Feb. 1960 Filling of the reservoir began.

Spring 1960 Borings S-1, S-2 and S-3 (see Figure 4) were drilled to depths of 172, 71 and 105 m, respectively, in the toe of the western side of the slide under the supervision of F. Giudici and E. Semenza.

Trenches were excavated in the depression south of the Pozza. Heavily fractured and highly permeable (water circulation was frequently lost) green and pink marly-calcareous materials were found towards the bottom of the borings. No traces of the Dogger and Malm formations and of the old sliding plane were found in these borings. The borings could not be drilled any deeper because of the continuous collapse of the borehole walls.[12] In the trenches, well-stratified cherty limestones with open cracks were found.

March 1960 The reservoir was filled to el 595 m. Small rockfalls took place just east and west of the Massalezza Ditch.

May 1960 The first survey reference points were installed on the left slopes of the Vaiont Valley.

June 1960 Giudici and Semenza submitted their formal report that established the presence of "numerous intercalations of greenish clay, with thickness of a few centimeters" in the Lower and Upper Cretaceous material of the site.[7] The presence of the various mylonitic zones in the left slope, particularly a mylonitic zone below the mouth of the Massalezza at an approximate elevation of 625 m, together with the rock debris remaining on the right valley slope was considered by Giudici and Semenza as evidence of an ancient slide in the left slope. They pointed out that the whole mass below Pian del Toc, between Casera Pierin and Colomber, could slide if the surface of the prehistoric slide was inclined towards the lake. Furthermore, they indicated this movement could be produced by the reservoir filling.

July 1960 Dal Piaz submitted another geological report in which he reexamined the stability of the reservoir banks. He found no evidence of past movement in the rock of the left bank. A similar large occurrence in the future was not considered possible. The report mentioned the possibility of smaller slides developing in loose layers near the surface between Pineda and the Pian della Pozza only. Partial and localized detachments of

continued

level increased, the piezometer gradients towards the reservoir would tend to be maintained in such a manner as to transport the same amount of water through the bedrock. Therefore, as reservoir levels increase, the piezometer pressures should also increase, causing progressively smaller amounts of rain to produce unstable conditions.

Precipitation records from three stations, covering the period from 1960 through 1964, in the vicinity of the Vaiont Dam were examined. Two of the stations, Erto and Cimolais, are located east of the dam at el. 726 and 652 m, respectively. The third station at Longarone is at an approximate elevation of 474 m. A study of the precipitation records reveals

rock "glebes and slices" along the edge of the Pian della Pozza, which would not extend to the Pozza itself, were predicted. "It was finally admitted that such detachments would help the area to reach a sure equilibrium" (p. 160).[20]

Summer 1960 Studies by E. Semenza were made to define the boundary of the old slide mass.

Sept. 1960 The dam was completed.

June - Oct. 1960 The reservoir level was raised from 595 to 635 m. Movements were recorded on the slope along the canyon wall from the dam to 350 m west of Massalezza Ditch, the region of the November 1960 slide noted below.

October 1960 The reservoir is filled to el. 635 m. Benchmark movements accelerated and a crack over 2 km long (see Figure 4) formed in the approximate location of the perimeter of the October 1963 slide. Approximately 500 mm of rain, the largest rainfall in the life of the reservoir, was measured at the Erto Station during this month. Cumulative movements of the sliding mass measured between the Massalezza Ditch and the dam exhibited an average of 1.0 m and a maximum of 1.4 m.[46]

Nov. 4, 1960 With the reservoir at el. 645 m, a 700,000 m^3 slide occurred on the left side of the valley just upstream from the dam (see Figures 4 and 20). This collapse produced a 2 m high wave in the reservoir.

Nov. 8 - 16, 1960 Müller, E. Semenza, Broili and others were called to Vaiont to investigate the movements of late October and early November 1960. In his Report No. 15, Müller outlined the nature of the movements, the various causes responsible for the movements and suggested a series of potential remedial measures.[41] He concluded that the sliding mass followed basically two types of movements: (a) a glacier-type movement that took place at the lower part of the slope between the dam and the Massalezza Ditch, and (b) a rigid block ("en block") type of movement that took place in the rest of the slide. He also concluded that it was not possible to stop the movements completely, and the only alternative was to maintain the slide under control by limiting the size of the sliding mass as well as the velocity of displacements. It was assumed that slow and controlled mass displacements would eventually build a passive resistance at the toe of the slide large enough to provide equilibrium. To gain control of the sliding movement, he recommended: (a) a slow and controlled lowering of the reservoir level, and (b) lowering and leveling of the phreatic level by means of two drainage tunnels driven underneath the sliding mass. These adits would start in the vicinity of the Massalezza Ditch at an approximate elevation of 900 m and run east and west, respectively. (Note, it is now known that the 900 m elevation was above most of the slide mass.) Other remedial measures — such as rock removal to reduce the weight of the "driving" mass, cementation of the sliding plane to improve the friction resistance along the sliding plane, and attempts to stop or considerably reduce the amount of water infiltrating the sliding mass — were considered either too expensive or be-

that the upstream stations of Erto and Cimolais (see locations in Figure 1) recorded much more precipitation that the station at Longarone. The records also show reasonably similar rainfalls at Erto and Cimolais. Since the Erto station was closest to the Vaiont Slide, its records were chosen to represent precipitation at the slide. However, actual precipitation on and above the slide very likely may have been higher than at Erto. Müller summarized the precipitation records in 10-day increments (shown at the top in Figure 36).[20]

In the winter months (November to March/April), the correlation between precipitation and slide movement would not be expected to be as reliable as when the precipi-

yond human endeavor.

Nov. - Dec. 1960 The reservoir is lowered from 650 to 600 m; slide movements were reduced.

Dec. 1960 Caloi completed a second seismic investigation. This investigation was more extensive than the first and included two traverses from an elevation of 750 m to the perimetral crack at the top of the slide. One traverse was about 200 m west of the Massalezza Ditch, the other about 400 m east of the ditch. This survey coincided with the 1959 survey in only one location along the 1.8 km length of the two traverses (see Figure 4). This time an upper layer of loose rock 30 to 50 m thick was found in the eastern part and a similar layer 70 to 150 m thick was found in the western part. Caloi concluded that there had been a deterioration in the rock quality since his first survey.[20]

Late 1960 Hydraulic model studies of slide induced reservoir wave phenomena were requested by C. Semenza.[12] Total slide movement ranged from .6 m to 1.5 m.

Early 1961 Exploration adits were driven in the Massalezza Ditch at about el. 920 to 950 m (see Figure 4).

April 1961 Broili and Weber visited exploration adits. They determined that the lower portion of the moving mass was at the contact between the Dogger and the Malm formations. They also determined that the movements did not take place along a single plane, but rather along a series of planes (passing through the fractured material) with clay layers sandwiched between solid pieces of rock.[46] Semenza indicated that, during the course of numerous visits to the adits, it was possible to determine that after a few tens of meters inside the underground openings, the loose materials present at the entrance changed into fractured rocks with folded stratification. Further on into the adit, after a section where a series of ultramylonitic facies were present, a sound uniformly bedded rock was found.[12] These strata dipped approximately 30° to 40° north and apparently represented the undisturbed beds beneath the zone of failure.

Early 1961 Bench mark system was extended over the total area included in the October 1960 movements.

Feb. - Oct. 1961 A bypass tunnel, shown in Figure 4, was constructed on the right bank to regulate the reservoir level in the Erto area in the event of a slide that would divide the reservoir. The reservoir was held down between el. 585 and 600 m during this period.

Sept. - Oct. 1961 Piezometers P1, P2, P3 and P4 were installed under the supervision of E. Semenza and F. Giudici (see Figure 4).

Oct. 1961 Carlo Semenza, designer of the dam, died.

Oct. 1961 - Feb. 1962 The water level in the reservoir was raised from 590 to 650 m. Mass movements during this period were almost negligible and the speed of movement remained below 0.1 cm/day.[46] The water level in the reservoir reached 635 m elevation in December 1961, the level at which the October 1960 movements and perimetral crack that outlined the 1963 slide developed. Movements were very small.

Oct. 1961 - Sept. 1963 Studies of

continued

tation was all rainfall. With much of the precipitation being snow, there would be almost no immediate infiltration; also, spring melting would cause large amounts of infiltration even though there might be no precipitation.

Graphical representation of rainfall and reservoir data is shown in Figure 37 on page 112. The water level in the reservoir is plotted against the amount of precipitation measured during the period 30 days preceding the arrival of the reservoir at those elevations. Similar plots were made for periods of 7, 15 and 45 days. It was believed that the "rain period" affecting the uplift pressures along the sliding plane would be bracketed between these intervals, although

the slide were generally limited to routine monitoring of slide movements and observations of groundwater levels.

Feb. 1962 - Oct. 1962 The water level in the reservoir continued to rise from 650 to 695 m. Around the first of October, when the water level in the reservoir was at an elevation of 695 m, the maximum speed of movement was still below 1 cm/day.[46]

April 20, 1962 Dal Piaz died.

Nov. 1962 The water level in the reservoir was raised to el. 700 m. Records indicate heavy rainfalls, 414 mm, during this month (230 mm in the first ten days) and the rate of movement increased up to 1.2 cm/day.[46]

Dec. 1962 - March 1963 The reservoir level was lowered very slowly to an elevation of 650 m. By the middle of February 1963, the reservoir level was at an elevation of 675 m, and the maximum rate of movement was 0.3 cm/day.[46] By the end of March, the reservoir level was at 650 m and the movements were almost nil. At the end of this period, the slide between the Massalezza Ditch and the dam had moved approximately 2.3 m. Bench marks at points of maximum displacements indicated total cumulative movements approximately equal to 3 m.[46] East of the Massalezza, the magnitude of movements was smaller.

April - May 1963 The reservoir level was raised from an elevation of 650 m to 696 m. Bench marks indicated a slight increase in the rate of movement up to 0.3 cm/day.[46]

June - July 1963 The water level in the reservoir had reached an elevation of 705 m by the middle of July. The maximum rate of movement measured at this time remained below 0.5 cm/day.[46]

Aug. - Sept. 1963 The water level in the reservoir was raised from 705 to 710 m elevation between mid-August and early September. Heavy rainfalls were measured in the middle of August (close to 200 mm between August 10 and 20). Unusually heavy rainfall (200 mm) was also measured in the following 20 days.

Sept. 1963 The rate of movement increased during the first days of September, while the reservoir level was slowly rising to a level of 710 m. The rate of movement reached values similar to those reached in October 1960 and in November 1962. By the middle of September, the maximum rate of movement at the lower west portion of the sliding mass reached a value of 3.5 cm/day.

By the end of the month, the maximum rate of movement of 3.25 cm/day was measured at points located in both the upper and lower parts of the western portion of the slide mass.[46] A slow drawdown to minimize the rate of movement of the sliding mass was started during the last days of September. Rainfall records at Erto indicate 164 mm of rain during this month.

Oct. 1 - 9, 1963 A drawdown of the reservoir level continued at the rate of about 1 m per day. Records indicated relatively heavy rainfalls of 29 and 22 mm on October 3 and 4. The rate of movement increased during the first days of October. According to the Bozzi Commission, the rate of movement on October 9 reached a value of 20 cm/day.[45] The total movement of the slide mass at this time was reported

longer term climactic effects could also be significant. The solid and half-filled dots on Figure 37 correspond to those occasions where accelerating movements exceeded 0.5 cm/day. The solid line through the lower range of these points would represent a "failure envelope" corresponding to those combinations of water level and precipitation required for the slope to become unstable. The extremes of this failure envelope, if extended, would correspond to:

- the reservoir elevation that would develop enough uplift pressure to make the slope unstable without any rainfall or snowmelt (approximately 710 to 720 m)

to have been between 3 to 4 m.

Oct. 9, 1963 A five-member board of advisors formed by the Italian Government in 1962 was evaluating conditions on a day-to-day basis. Prof. Penta, the geologist member, was scheduled to visit the slide area on Oct. 10 (Kiersch, personal communication to the authors).

At 10:39 p.m., with the reservoir level at el. 700.4 m, the Vaiont Slide took place.[45]

Oct. 10, 1963 The Minister of Public Works appointed an inquiry commission with Carlo Bozzi as chairman. The other members were: Engr. Giuseppe Merla, and Professors Livio Trevisan, Raimondo Selli and M. Viparelli. Their report was submitted January 16, 1964.[45]

E. Semenza made his first visit to the site after the slide. An extensive geological study of the slide mass and surrounding areas was undertaken by Semenza and Rossi for ENEL.

Oct. 23 - 27, 1963 Müller and Broili, together with Engr. H. Maier and Prof. G.A. Kiersch, made their first visit to the site after the slide. Extensive investigations for ENEL began, continuing into 1964.

Nov. 1, 1963 A commission was appointed by ENEL to "ascertain the causes of the Vaiont disaster." The members of this commission were: Avv. Marcello Frattini, and Professors Filippo Arredi, Alfredo Boni, Costantino Fasso, and Francesco Scarsella. Prof. Filippo Falini, also a member of this commission, died in a helicopter accident in the Vaiont area while investigating the slide in November. The commission was commonly known as the "Frattini Commission" and submitted its report in January 1964.[39]

- the rainfall or snowmelt required to make the slope unstable without the reservoir present (approximately 180 mm/7 days, 350 mm/15 days, 700 mm/30 days and 1,100 mm/45 days)

Various combinations of reservoir elevation and preceding precipitation that correspond to different situations during the lifetime of the reservoir (impoundments as well as drawdowns) are represented in Figure 37. As indicated in the figure, the combinations represented by open triangular points correspond to relatively stable conditions. Open circles indicate when the rate of movement is less than 0.5 cm/day. These points plot generally below the failure envelope and there is a tendency for the rate of movement (given by the number in the parentheses) to increase for those combinations closer to the failure envelope.

The two main variables affecting the stability of the slide were the reservoir level and the amount of precipitation in the preceding period. Figure 37 suggests that the slide would have failed with no rainfall or snowmelt when the reservoir levels reached the vicinity of the proposed full supply level, el. 722.5 m. The data also indicate that slide movements could be triggered by very high rainfalls or snowmelts, the magnitude of which were in the range of 130 to 200 percent of the 7- to 45-day precipitations recorded for the period from 1960 to 1964. Therefore, slope movements have most certainly resulted from the maximum precipitation occurring within 100-year to 1,000-year intervals in the past. Correlation with precipitation data would appear to provide quantitative verification of the stories attributed to the local inhabitants about the occurrence of slope movements from time to time.

Assumptions for Stability Analyses

The shear strength along the base of the slide was assumed to be related more to the residual shear strength of the multiple layers of clay found along the basal surface of sliding than to the higher shear strengths of the rock-to-rock contacts. This assumption represents a basic departure from previous stability analyses, such as those performed by Müller, Lo *et al.*, Chowdhury and others.[4,20,42,6] The bases for this assumption are the authors' field observations, summarized in Table 1, and the results of the laboratory shear strength and Atterberg limit tests, summarized in Tables 2, 3, 4 and 5, and in Figure 35.

Essentially, all peak strengths and most

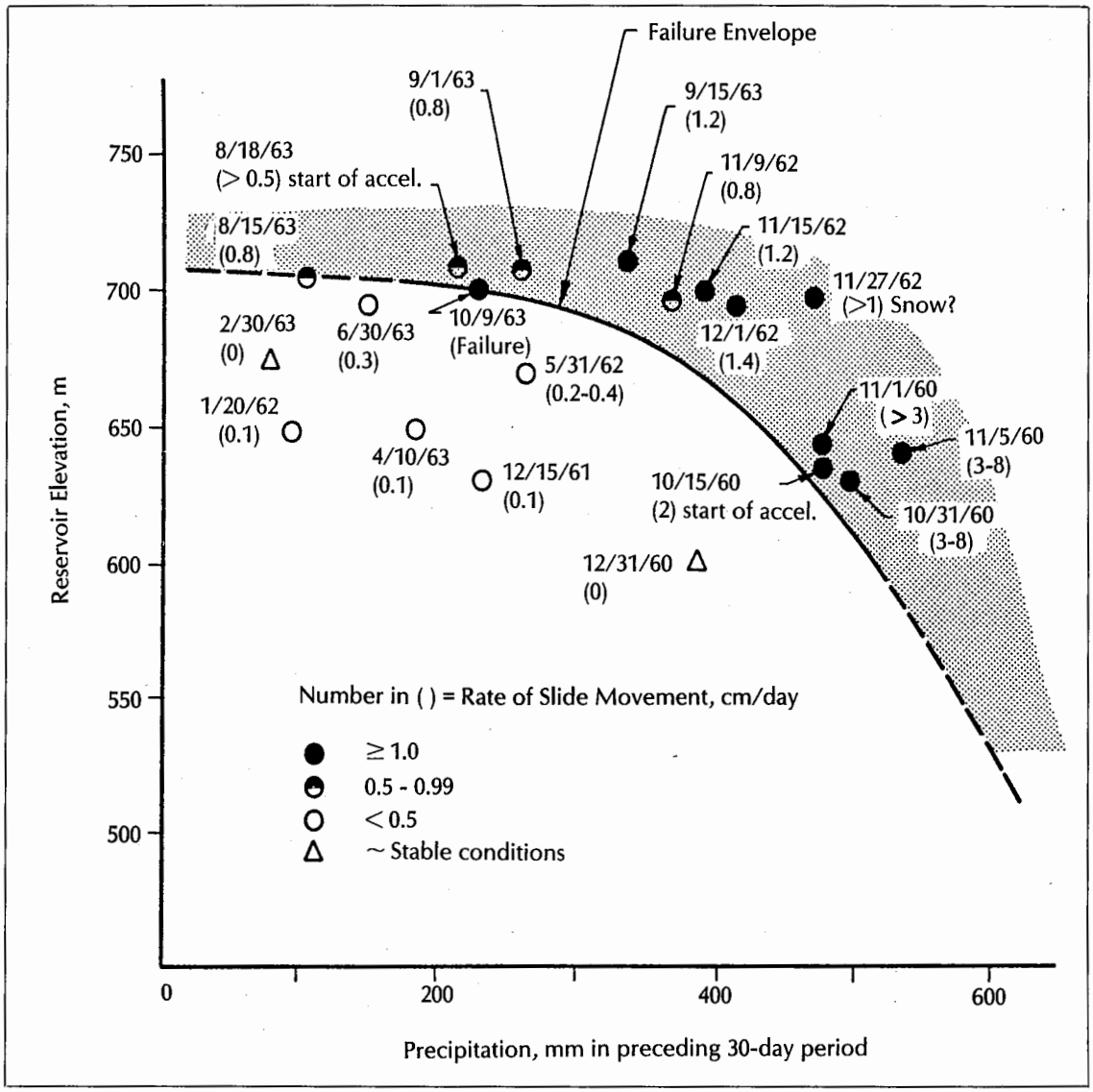


FIGURE 37. The stability of the Vaiont Slide for reservoir elevation *vs.* 30-day precipitation.

increases in strength caused by irregular geometric effects were assumed to have been lost because of prehistoric slide or tectonic movements. Thus, the residual strength, ϕ_r, of the basal failure plane materials was assumed to be the most significant factor in these analyses. However, modest strength increases, from those indicated by the results of laboratory residual strength tests of the weakest clays, could be expected because of some rock-to-rock contacts that occur along the basal sliding surface. These contacts occur because of the existence of:

- localized areas of shearing across bedding planes,
- areas where clays do not occur, and
- areas where rock beds are brought into contact because clays could be squeezed and forced to flow into voids that develop as a result of the displacement of adjacent irregular rock surfaces.

Also, small increases in shear strength could be expected as a result of the introduction of brecciated rock fragments into the clays along the surface of sliding.

The residual angle of shearing resistance, ϕ_r, of the clays as determined from the laboratory tests ranged from 5° to 16°. Most values fell within a range of 6° to 11°, and, for whole samples at high stress levels, the results were between 8° and 10°. However, because of the factors noted above, it seems quite reasonable to accept a mean value for ϕ_r along the basal surface of sliding of 10° to 12°. Cohesion is, of course, assumed to be essentially zero. The basal surface of sliding is assumed to correspond to the old rupture surface, as described by Semenza.[35] The surface of sliding may also be coincident with a tectonic thrust fault. Previous assumptions are in agreement with the occurrence of an uncemented basal fault or a pre-existing rupture surface.

In addition to the shearing that occurs along the slide's base, deformations occur within the slide mass as it moves over irregularities and across gross changes in inclination of the failure plane. In a highly disturbed rock mass, such as the Vaiont Slide, much if not all of these deformations will occur along pre-existing discontinuities, such as joints and faults. In this study, the angle of shearing resistance, β, acting along the discontinuities that cross bedding planes within the slide mass, was chosen on the basis of the authors' experience and field observations of the type of materials present. It was apparent that deformation along these planes would require shearing across thinly bedded limestones, cherts and clay interbeds of the Lower and Upper Cretaceous formations. On a large scale, these beds can deform locally and would be expected to develop an angle of shearing resistance, β, of 30° to 40°. This potential shearing resistance is not mobilized except where there is a tendency for adjacent slices to move relative to each other as described by Mencl.[47] With the geometry of the failure surface established, the largest amount of relative movement of slices would occur at the junction between the "back" that dips at 25° to 45°, and the nearly horizontal "seat" of the slide. Analyses have indicated that the stability of the slide is sensitive to the value of β.

Since there is no practical way to measure β in the field, the analytical procedure was arranged so that the effect of various assumed values of β could be determined. The β values along the discontinuities are assumed to be somewhat higher than the values of the residual shear strength along the basal surfaces due to the fact that these surfaces have undergone fewer differential movements than those occurring along the slide base. Therefore, some additional strength losses could presumably occur with continued displacement along the near-vertical surfaces.

In the three-dimensional analyses undertaken here, it is necessary to assume that differential movement may also occur between adjacent blocks. The angle of shearing resistance that can be mobilized along the sides of the blocks (those near-vertical planes oriented parallel to the direction of slide movement) within the slide mass is assumed to be similar to the values used for β since the materials are the same.

The frictional resistance of the steeply dipping planes forming the east end of the slide was assumed to be 36°. These values are in agreement with published data on the residual angle of shearing resistance for carbonate rocks.[40]

The piezometric head acting on the surface of sliding along its contact with the reservoir was assumed to be equal to the reservoir level. Away from the reservoir, the piezometric head was assumed to increase above reservoir levels due to an assumed groundwater flow system where water was moving from the mountain towards the valley. The initial water level recorded in drillhole P2 in October 1961 was over 90 m above the reservoir level. This reading is a control point on the fluid-pressure distribution curve for the "low rainfall" condition. For the sections without P2, an equivalent pressure difference was assumed at P2 than what was measured. For both low and high rainfall conditions, the difference between the assumed piezometric level at any point on the failure surface and the elevation of that point is gradually reduced to zero between the location of P2 and the southern extremity of the slide surface. The actual piezometric levels used in the analyses are given in a previous

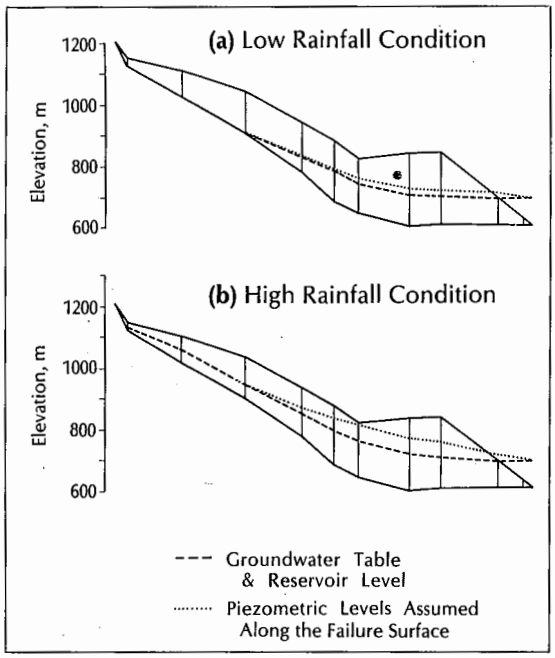


FIGURE 38. Piezometric levels used in stability analyses for Section 2, at a reservoir elevation of 710 m.

report.[18] An example of the actual piezometric levels used is illustrated in Figure 38 for Section 2 with a reservoir level of 710 m.

Each reservoir condition was analyzed for a low and a high piezometric pressure distribution corresponding to a low and a high rainfall condition. The actual pressure distributions would vary for intermediate rainfalls. For each reservoir level, the calculated factor of safety would be expected to differ from its real value. The amount of the correction from a known value, say at failure, would be an indication of the real fluid pressure distribution due to intermediate rainfalls, if other factors remained constant.

Figure 38 shows that the groundwater table above the clay layers was assumed to equal the reservoir level at the toe of the slide and to slope gently upwards to agree with the water levels measured in P1 and P3 (and in P2 after July 1962). The water pressure distribution on the ends of the vertical slices was assumed to be consistent with a hydrostatic increase in water pressure within the slide debris.

These interpretations of the water pressure distributions in the slide are in agreement with the only quantitative data available (*i.e.*, the water levels recorded in P1, P2 and P3) and account for the entire history of the water levels observed in P2.

The base of the slide has been assumed to correspond to a prehistoric slide surface. The position of the failure surface toe was assumed to agree with the contact along the Vaiont Canyon given by Giudici and Semenza, and Rossi and Semenza in their maps of the pre-slide geology.[7,26] The stratigraphic sequence of Rossi and Semenza was used to plot the base of the slide between exposures currently visible along the back of the slide and along the toe.[26] Location of the failure surface is further based on all available post-slide drillhole data including the interpretations by Broili, Rossi and Semenza, and more recently (by Rossi and Semenza) in the preparation of sections used in this study.[2,26] Although the core recovery was poor in the slide material and in the thinly bedded Lower Cretaceous and Malm units associated with the failure surface, it was generally possible to recognize the top of the Dogger formation with confidence. The failure surface was assumed to be located at a constant distance above the top of the Dogger formation, except near the eastern boundary. An offset distance above the top of the Dogger was similarly selected to establish the failure surface position along east-west sections (for example, see Figure 27).

The sections used for the two-dimensional analyses in the first stage of the overall reevaluation were drawn approximately parallel to the direction of the initial movement of the slide as determined by survey records. The orientation of these sections is shown in Figure 4.

The combination of low shear strength on the slide base and the pronounced eastward (upstream) dip of the failure surface along the base of the seat of the slide required that the shearing resistance developed along the east side of each north-south slice of the slide be considered in the analyses. In particular, this assumption was necessary to demonstrate the relative stability of the slide prior to reservoir filling. The step-like shape of the

eastern side of the basal failure surface corresponds to surface observations of the tectonic fault at several locations. The base of the slide was assumed to be relatively continuous and sub-parallel to the bedding across the seat and the back of the slide, although the basal plane is assumed to step upwards as the eastern limit of the slide is approached.

Large movements were assumed to have occurred along a pre-existing surface of rupture. These movements occurred in an alignment sub-parallel to the direction of slide movement. Significant downhill movements along the failure plane are assumed to have occurred periodically during valley erosion and glacial loading and unloading in Pleistocene times. One of these movements was probably rapid and was responsible for the large remnant of slide material mapped by Giudici and Semenza on the right side of the valley prior to the 1963 slide (see Figures 4 and 7a).[7] Additional movements of the slide mass have occurred in post-glacial times with the erosion of the most recent Vaiont Canyon through the slide mass and the removal of support from the toe of the slide. These movements continued into historic times and are probably the source of the tales told by local residents about the instability of the slope and the name Mt. Toc, which is reported to mean "crazy" in the local dialect. Talus of various ages infills a large zone along the base of the top scarp of the slide, providing evidence of historic and prehistoric movements. Also, the aligned depressions and airphoto lineaments that can be noted along the perimeter of the slide and within the slide in the 1960 airphotos (see Figure 32) clearly reflect such prehistoric slide movements.

Most of the slide, including both sides of the Massalezza Ditch, is assumed to have moved as a unit to the northeast. Previous analyses have generally assumed a difference in slide movement and other factors on either side of the Massalezza Ditch.[4,20] The authors' field observations and interpretations of the pre-slide and post-slide mapping of Rossi and Semenza indicate that there was one general direction of slide movement and that the main slide mass did not undergo significantly different directions of movement.[26] This evidence also indicates that a much smaller slide from the east side came down onto the main slide. This secondary slide was probably triggered by loss of toe support produced by the movement of the main slide. However, consideration of this secondary slide is not necessary for an understanding of the main slide.

Sections 2, 5 and 10A (see Figure 4) were considered for analysis since they were in the approximate direction of the initial slide movement as determined by displacement vectors calculated from measurements of survey monuments.

Stability Analyses

The purpose in making stability analyses after a slide has occurred is to develop a more complete and more quantitative understanding of the factors that led to the slide. Such analyses also provide a check on the principal input parameters — including the shear strength along the slide surface, the geometry of the failure surface and the distribution of water pressures along the surface of failure — that had an effect on the slide. For stability analyses conducted after a slide, the geometry of the failure surface is determined by pre- and post-slide drillhole data and geologic mapping. Water pressure distributions may be estimated from pre-slide piezometric observations and geohydrologic interpretations. The shear strength data used may be based on laboratory tests or assumptions. In many cases, the shear strength is back-calculated for the failure condition, assuming a factor of safety of 1.0 at failure and assuming the geometry and pore pressures are known values.

Post-slide stability analyses are, in fact, a quantitative means of verifying the story developed to explain the slide. For example, when significant rates of movement were recorded at various times during the history of slide movement, various analyses should yield calculated factors of safety very near 1.0. It is equally important that the stability calculations yield factors of safety appreciably greater than 1.0 for those times in the slide history when the movements were known to be insignificant. If the results of the analyses do not agree with all the available data and

TABLE 7

Results of Previous Stability Analyses, from Müller[4]

Author*	Cross Section (See Fig. 34)	Reservoir Water Level	Inclination of the Water Level	$\phi_{req.}$	$tg_{\phi_{req.}}$
Mencl (1966a)	2	700	—	18.75	0.339
Mencl (1966b)	2	700	—	20.5	0.364
				17.5	0.316
				18.5	0.325
Kenney (1967)	2	600	—	19.4	0.352
		650		20.1	0.366
		700		20.7	0.378
		600		21.8	0.400
		650		22.0	0.404
		700		22.2	0.408
Nonveiller (1965b)	2	590	~10°	22.1	0.406
		650	~4°	22.1	0.406
		700	2°	24.0	0.445
Nonveiller (1966a)	2	700	2°	27.7	0.525
	differing			27.0	0.510
	very much			28.5	0.542
Müller (cal.	2	600	—	21.0	0.384
according to		650		21.8	0.400
Mencl [1966b])		700		22.5	0.414
(according to		600	—	20.4	0.372
Kenney [1967])		650		21.2	0.388
		700		21.9	0.402
(according to		600	~10°	18.8	0.340
Nonveiller [1965b]		650	~4°	20.1	0.366
but considering		700	2°	20.8	0.380
the actual shape					
of the slip					
surface accord.					
to Broili [1967])					

*References given in bibliography in Müller[4]. **Description of premises: (1) c =0; (2) tg ϕ has the same value along the whole slip surface (no zone has a higher shear resistance); (3) stiffness of the slip mass is not considered; (4) secondary failures are not considered; and (5) hydrodynamic pressure is not considered.

with the observed movement record, then the explanation developed is incorrect or at least incomplete. Because of the importance of the Vaiont Slide as a precedent, it is essential that stability analyses be in agreement with observed facts.

Stability analyses are a much more powerful tool when there has been a definite condition of failure or a significant rate of movement because the factor of safety can then be assumed to be 1.0 and various combinations of shear strength and pore pressure distribution can be investigated that will yield a factor of safety of 1.0. If several periods of movement have occurred under differing reservoir conditions, it is possible to further eliminate some of the ambiguity in the input to the stability analyses. Such is the case for the Vaiont Slide where four periods of movement have been identified. None of the analyses by Mencl, Kenney, and Nonveiller, which were summarized by Müller (see Table 7), or the later analyses by Kahn, Lo *et al.*, Jaeger, Trollope, and Chowdhury explain the known movement record for these periods.[47,14,16,17,4,48,42,49,5,6] Also, these analyses did not resolve the conflict between the unstable behavior observed in October 1960 when the reservoir was at el. 650 m with unstable behavior exhibited by the slope when the reservoir was at el. 650 m during January 1962.

The analyses performed for this study were designed to examine the equilibrium conditions of the Vaiont Slide for three periods when the factor of safety was near 1.0:

- prehistoric times, when geologic field evidence indicates that movement had occurred;

Calculation According to*	Assumed or Neglected	Premises Tacitly Assumed or Neglected**	Remarks
—	on secondary slip surfaces ϕ = 30° or 40° resp.; c = 50 t/m²	1 2 3 5	Prandtl's wedge
Petttersson		1 2 3 4 5	Prandtl's wedge
Mencl	(like Mencl [1966a])	1 2 3 5	Zone of arching
Mencl		1 2 3 5	
Janbu (1954)	—	1 2 3 4 5	Data taken from small drawings
Nonveiller (1965)	on the upper part of the slip surface is kept ϕ = 25° = constant	1 3 4	Assumptions of the slip surface position & form differ very much from the nature
Nonveiller (1967a)	same	1 2 3 4	
Pettersson	—	1 2 3 4 5	—
Janbu	—	1 2 3 4 5	—
Nonveiller	—	1 2 3 4	—

- October 1960, when the perimeter cracks developed; and
- the fall of 1963, when accelerating movements began just prior to October 9, 1963.

Two groundwater conditions were considered for each period, one representing periods of high rainfall and the other low rainfall. In addition, the case of a dry slide was included for control purposes. Differences in the behavior of the slide between October 1960 and January 1962 were expected to be explained by differences between the high and low rainfall groundwater conditions for a reservoir at el. 650 m.

Two-Dimensional Stability Analyses

The two-dimensional analyses conducted used a variation of the method of slices that is shown schematically in Figure 39. Developed at the request of the authors, this method is described by Anderson in Hendron and Patton.[50,18] The analyses of the three cross sections chosen as representative of the different portions of the slide were carried out by means of a computer program that calculated the factor of safety by considering the surface of sliding as a series of planes. Each cross section was subdivided into slices with vertical boundaries between slices as shown in Figure 39a.

Shear forces between slices were considered in these analyses. The maximum obliquity permitted for the resultant lateral force, F, acting on a vertical plane between slices is defined as β (Figure 39b) and is representative of the shearing resistance across strata of limestone, chert, siltstone and clay. The values of β used in these analyses

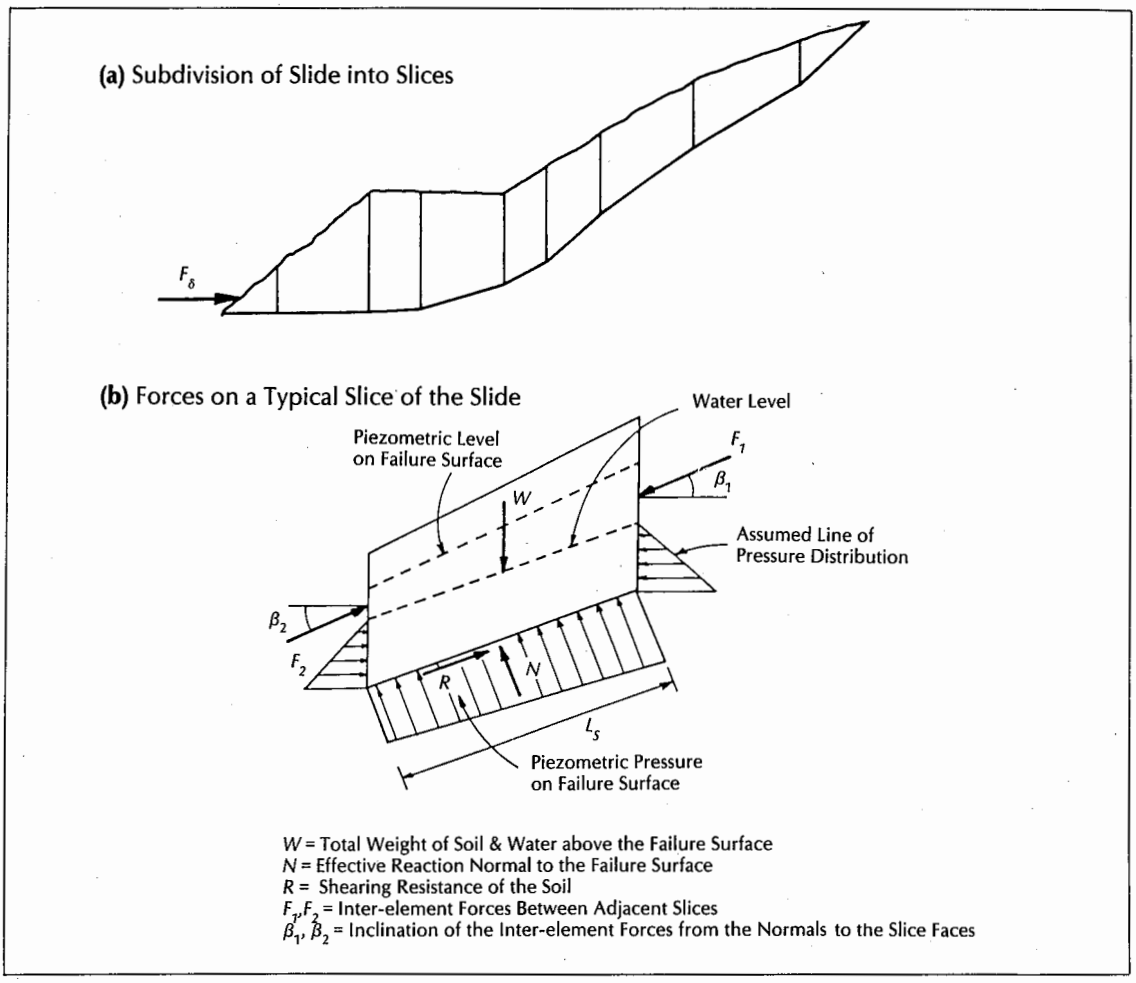


FIGURE 39. Selection of typical slices and forces acting on a typical slice.

ranged from 30° to 40°.

With the magnitude of the value β input to the program, the resultant effective forces between slices may be inclined at an angle β above or below the horizontal as shown in Figure 39b. The angle will depend on the relative changes in the slope of the base planes that support adjacent slices. The analyses satisfied the equations of horizontal and vertical equilibrium, but rotational equilibrium was not considered. The shear force resisting movement at the base of each slide, R, is formulated by:

$$R = \frac{(C' L_s + N \tan \phi')}{F.S.} \qquad (1)$$

where:

C' = cohesion (assumed to be zero in this analysis)

L_S = base length of the slice

N = "effective" normal base reaction

ϕ' = "effective" angle of shearing resistance

$F.S.$ = factor of safety

The computation involved the assumption of an initial factor of safety, and an iterative process was employed in which the factor of safety was changed until the slide mass was computed to be in equilibrium for all slices at the same factor of safety. The program also determined the horizontal force that would have to be applied to the downhill side of the lowermost slice to bring the slide to a factor of safety of 1.0. If the factor of safety

TABLE 8

Cases Analyzed for Sections 2, 5 and 10A

Case	Groundwater Level or Rainfall Condition	Reservoir Elevation, m
1	Low	None
2	High	None
3	Low	650
4	High	650
5	Low	710
6	High	710
7	No pore pressures on failure surface	

was less than 1.0, the force, designated as F_δ (Figure 39a), would be compressive. If the factor of safety was greater than 1.0, then F_δ would be a tensile force.

Cross sections 2, 5 and 10A were each analyzed for seven different cases that corresponded to different combinations of reservoir elevation and rainfall. These cases are summarized in Table 8 and include the case of no reservoir and instances when the reservoir elevation was at 650 and 710 m. For each reservoir elevation, both low and high groundwater levels in the slope are considered to account for low and high periods of rainfall. The low and high piezometric elevations along the failure surface were obtained from the piezometric elevations recorded in piezometer P2 in the fall of 1961. In addition, as a reference calculation for the cases cited in Table 8, each cross section was analyzed for the case of no pore pressure on the failure surface. The cross section and piezometric elevations considered for Section 2, cases 5 and 6, are shown in Figure 38.

The factors of safety calculated from the two-dimensional slope stability analyses for Sections 2, 5 and 10A are summarized in Table 9. For Sections 2 and 5, analyses were conducted for all seven water level conditions listed in Table 8 for ϕ values of 8°, 10° and 12°, and β values of 30° and 40°. For Section 10A, all seven cases were calculated for a ϕ value of 12° and β values of 30° and 40°.

An inspection of factor of safety values presented in Table 9 indicates that, even for no reservoir, the factors of safety are low for the shear strength used. If the values for ϕ = 12° and β = 40° are studied for the no reservoir case, Section 2 has a factor of safety ranging from 0.63 to 0.73, Section 5 has a factor of safety ranging from 1.04 to 1.18, and Section 10A has a factor of safety ranging from 0.51 to 0.57. In all cases, Section 5 is more stable than Sections 2 and 10A because Section 5 is closer to the Massalezza Ditch where the volume of material on the steep backslope is less than for Sections 2 and 10A.

It is also interesting to study the results for Section 5 for ϕ = 12°, β = 40° for cases 1 through 6. In comparing cases 1 and 2, the difference between high and low groundwater levels makes about a 14 percent change in the factor of safety for no reservoir. In comparing cases 1 and 5 with cases 2 and 6, there is a 12 to 14 percent change in the factor of safety caused by the reservoir changing from river level (450 m) to 710 m. A comparison of cases 5 and 6 shows that, at a reservoir elevation of 710 m, the difference in high and low rainfall could change the factor of safety by about 16 percent. Thus, it appears that for the unstable slope the changes caused by rainfall are just as significant as changes in the reservoir level.

Table 10 summarizes the calculated forces, F_δ, for Sections 2, 5 and 10A that are required to maintain the slide at a factor of safety of 1.0. The calculated forces, F_δ, are assumed to be applied horizontally to the lowermost slice.

Taken as a whole, the results of the two-dimensional calculations shown in Tables 9 and 10 indicate that the factors of safety are too low for the slide mass to have been stable over much of its history. Therefore, the shear strengths must have been higher, the pore pressures lower, or an important element has been omitted from the two-dimensional analyses. The pore pressure distribution assumed seems quite reasonable and the angle of shearing resistance of 12° is consistent with measured residual shear strengths, plus an increment to the angle of shearing resistance to account for local rock-to-rock contacts. It seemed reasonable, therefore, to check the effects of the three-dimensional nature of the

TABLE 9

Vaiont Slide, Calculated Factors of Safety

(a) Section 2

Case	$\phi = 12°$		$\phi = 10°$		$\phi = 8°$		Reservoir	Groundwater
	β=30°	β=40°	β=30°	β=40°	β=30°	β=40°		
1	.651	.728	.540	.604	.431	.481	none	low
2	.562	.632	.466	.524	.372	.418	none	high
3	.627	.699	.520	.579	.414	.562	650 m	low
4	.540	.605	.448	.501	.357	.399	650 m	high
5	.560	.621	.465	.515	.371	.410	710 m	low
6	.469	.520	.399	.431	.310	.314	710 m	high
7	.714	.801			no pore pressure on failure surface			

(b) Section 5

Case	$\phi = 12°$		$\phi = 10°$		$\phi = 8°$		Reservoir	Groundwater
	β=30°	β=40°	β=30°	β=40°	β=30°	β=40°		
1	.943	1.184	.782	.984	.624	.784	none	low
2	.838	1.038	.695	.863	.554	.688	none	high
3	.911	1.142	.756	.949	.602	.756	650 m	low
4	.801	0.991	.665	.822	.530	.656	650 m	high
5	.856	1.062	.711	.883	.566	.704	710 m	low
6	.738	0.899	.612	.746	.488	.594	710 m	high
7	1.144	1.505			no pore pressure on failure surface			

(c) Section 10A

Case	$\phi = 12°$		$\phi = 10°$		$\phi = 8°$		Reservoir	Groundwater
	β=30°	β=40°	β=30°	β=40°	β=30°	β=40°		
1	.530	.574					none	low
2	.471	.514					none	high
3	.514	.557	Not Run		Not Run		650 m	low
4	.456	.496					650 m	high
5	.470	.508					710 m	low
6	.410	.445					710 m	high
7	.604	.655			no pore pressure on failure surface			

slide surface before abandoning the assumed values of shearing resistance and pore pressure distributions.

Three-Dimensional Nature of the Slide Surface

Figure 40 is a schematic diagram that illustrates the three-dimensional nature of the possible failure wedges resulting from the upstream dip of the failure surface at the base of the slide. In this figure, the plane *a-e-d* is taken as a vertical plane in the direction of movement and would be parallel to Sections 2, 5 and 10A. The surface *a-d-c-b* corresponds to the basal bedding plane failure surface, the trace *a-b* represents the outcrop of the bedding planes on the wall of the Vaiont Gorge, and the trace *b-c* represents the western extent of the slide. The shearing force, τ_1, is the shearing resistance mobilized on the base plane parallel to the direction of the movement and is the resisting force calculated in conventional two-dimensional analyses. The shearing force, τ_2, is the shearing resistance mobilized parallel to the direction of movement on the vertical plane *a-e-d* due to the normal force PN_2. PN_2 is the supporting force required on plane *a-e-d* to prevent movement upstream down the apparent dip of the bedding surfaces in a

TABLE 10

Vaiont Slide, Force Per Unit Width Required to Maintain Equilibrium of the Slide

(a) Section 2 — units (N/lineal m of slide) x 10^7

Case	$\phi = 12°$		$\phi = 10°$		$\phi = 8°$	
	$\beta = 30°$	$\beta = 40°$	$\beta = 30°$	$\beta = 40°$	$\beta = 30°$	$\beta = 40°$
1	29.37	18.27	38.94	26.58	48.45	34.70
2	37.36	25.31	45.78	32.65	54.11	39.76
3	31.41	20.28	40.63	28.28	49.81	36.05
4	39.30	27.27	47.40	34.32	55.43	41.13
5	36.34	25.21	44.51	32.21	52.63	39.01
6	56.11	33.18	52.04	39.23	59.02	45.05
7	24.02	13.29	Dry Case		Dry Case	

(b) Section 5

Case	$\phi = 12°$		$\phi = 10°$		$\phi = 8°$	
	$\beta = 30°$	$\beta = 40°$	$\beta = 30°$	$\beta = 40°$	$\beta = 30°$	$\beta = 40°$
1	4.42	-10.10	16.68	0.86	28.73	10.61
2	12.74	-2.20	23.88	7.31	34.83	16.14
3	6.68	-7.63	18.24	2.52	29.60	11.60
4	15.12	0.53	25.71	9.26	35.94	17.41
5	10.26	-3.18	20.63	5.50	30.80	13.42
6	19.32	5.22	28.57	12.81	37.61	19.81
7	-11.04	-26.20	Dry Case		Dry Case	

(c) Section 10A

Case	$\phi = 12°$		$\phi = 10°$		$\phi = 8°$	
	$\beta = 30°$	$\beta = 40°$	$\beta = 30°$	$\beta = 40°$	$\beta = 30°$	$\beta = 40°$
1	62.26	47.65	—	—	—	—
2	70.82	55.24	—	—	—	—
3	63.12	48.53	—	—	—	—
4	71.42	55.98	—	—	—	—
5	66.65	52.23	—	—	—	—
6	75.52	60.30	—	—	—	—
7	52.39	38.44	Dry Case		Dry Case	

Note: $\beta = 0$ between the toe element & the next uphill element for all runs on Section 5.

direction perpendicular to plane *a-e-d*. The resisting force, τ_2, as shown schematically in Figure 40, is shown below to be significant and necessary for equilibrium of the slide at all times. This requirement would be true even before the filling of the reservoir, assuming the shear strength along the bedding planes was governed by the clays and was in the range of 8° to 12°. The angle of shearing resistance used along the base of the slide *a-b-c-d* was 10° to 12°. The angle of shearing resistance on planes parallel to plane *a-e-d* in Figure 40 was assumed to be ~ 36°.

Figure 41 illustrates a cross section taken at right angles to the two-dimensional sections identified by Sections 2, 5 and 10A. The triangular wedge *a-e-b* in Figure 41 corresponds to the east-west section shown as *a-e-b* in Figure 40. The surface shown as *b-a* in Figure 41 represents the eastward dipping failure surface as shown in Figure 27. The total horizontal normal force PN_2 required in section *a-e*, Figure 41, to prevent upstream (eastward) movement down the apparent dip of the bedding planes is formulated by:

$$PN_2 = W \tan \theta \qquad (2)$$

where:

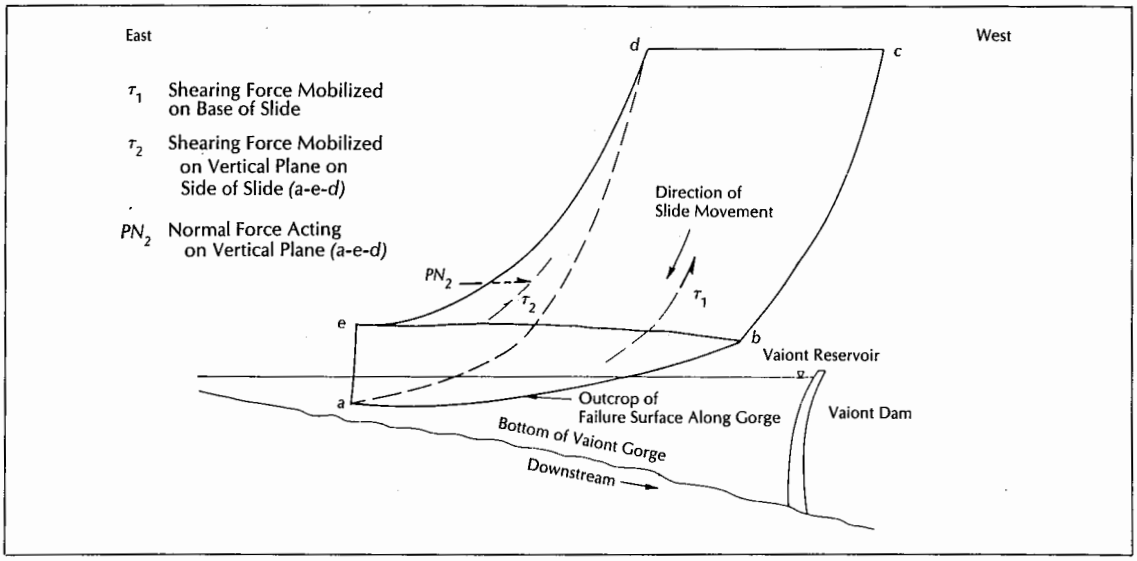


FIGURE 40. A schematic illustration of the three-dimensional nature of the Vaiont Slide mass.

W = weight of the slide mass to the west of the cross section being considered

θ = average upstream dip of the sliding plane in a direction perpendicular to planes containing Sections 2, 5 and 10A

The effective normal force $\overline{PN}_2$ is then equal to:

$$\overline{PN}_2 = PN_2 - U_h \tag{3}$$

U_h = hydraulic force against the face of the cross section

The frictional force acting parallel to the face of the cross section can then be calculated as:

$$\overline{PN}_2 \tan \phi_R \tag{4}$$

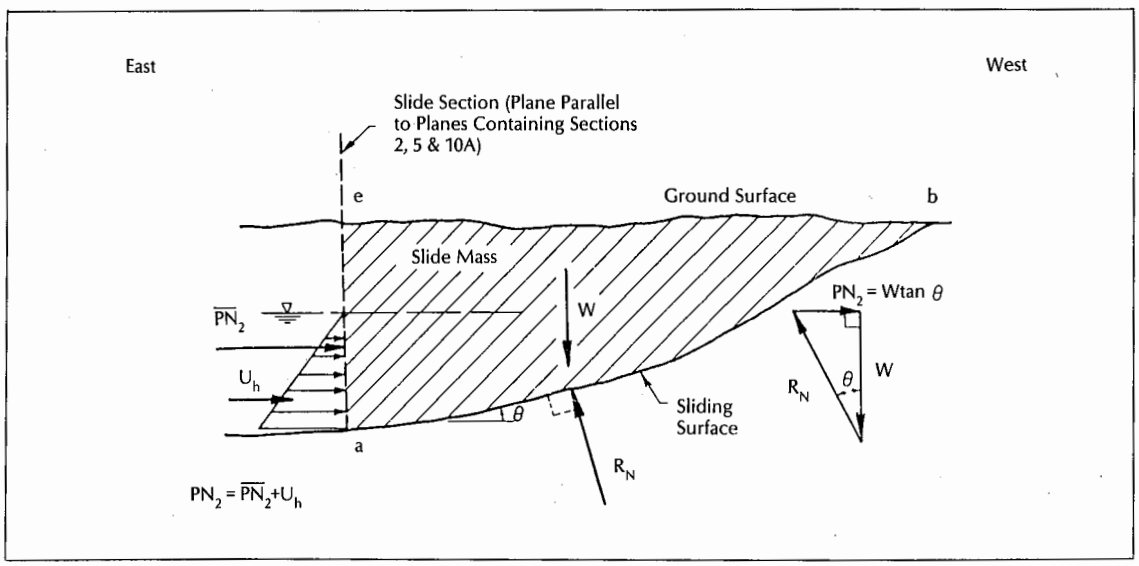


FIGURE 41. Representation of the normal forces on a slide cross section developed by the upstream dip of the sliding surface.

where:

ϕ_R = the friction angle along the vertical surface between adjacent cross sections

The frictional force on the slide plane *a-b* (Figure 41) does not have a component downhill along *a-b* perpendicular to Sections 2, 5 and 10A. The frictional force in the bedding plane base of the slide is parallel and opposed to the direction of movement; therefore, it is parallel to planes 2, 5 and 10A.

Factors of safety of the cross sections considered to be representative of the sliding mass under the seven different conditions investigated were calculated. Stability analyses of the three cross sections were carried out with the modified slice method described above. These analyses resulted in the factors of safety and equilibrium force, F_δ, per unit width of the slope given in Tables 9 and 10. The resisting force along the failure plane and the driving force acting on each unit width of slide represented by that particular cross section were then estimated as:

$$\frac{\textit{Resisting force}}{\textit{Driving force}} = \frac{\sum (N_i) \tan \phi}{\sum W_i \sin \alpha_i} = F.S. \qquad (5)$$

where:

W_i = weight of each slice in the cross section

α_i = angle of inclination of the bottom of the slice

ϕ = effective angle of friction along the failure plane.

N_i = effective normal force at the base of slice *i*

When the factor of safety is equal to 1.0 with the equilibrium force acting, Equation 5 becomes:

$$\frac{\sum (N_i) \tan \phi + F_\delta}{\sum W_i \sin \alpha_i} = 1.0 \qquad (6)$$

Equations 5 and 6 result in:

$$\frac{F_\delta}{\sum W_i \sin \alpha_i} = 1.0 - F.S. \qquad (7)$$

where the driving force is formulated by:

$$\sum W_i \sin \alpha_i = \frac{F_\delta}{1 - F.S.} \qquad (8)$$

and the resisting force along the failure plane is derived from:

$$\sum (N_i) \tan \phi = \frac{F_\delta}{\frac{1}{F.S.} - 1} \qquad (9)$$

Equations 8 and 9 were then used to calculate the resisting as well as the driving forces for various sections of the sliding mass. The total driving, resisting and restoring forces acting on the entire sliding mass were then obtained from the product of each force per unit width, times the width of the slope represented by the typical cross section where those forces were calculated. Factors of safety of the entire mass, including the frictional force along the eastern wall boundary, were then redefined as:

$$F.S. = \frac{\sum (N_i) \tan \phi + \overline{PN}_2 \tan \phi_R}{\sum W_i \sin \alpha_i} \qquad (10)$$

Calculated values of this redefined factor of safety of the entire mass were carried out in Hendron and Patton for the different water elevations considered in this study and are listed in Table 11.[18]

A friction angle of 12° along the failure surface was considered to be the most representative of the in-situ materials at the slip surface. The friction angle, ϕ_R, along the eastern wall boundary where displacements took place between rock surfaces (as indicated by the traces in the exposed wall) was estimated to be 36°. The friction angle, β, along vertical rock surfaces between slices used in the calculation of these revised factors of safety was taken as 40°.

As the values in Table 11 indicate, failure (factor of safety equal or near to 1.00) would occur under the combined effect of a heavy rainfall (developing a high groundwater level) and a reservoir elevation of 710 m. The slope

TABLE 11

Factor of Safety of Sliding Mass Calculated From Three-Dimensional Stability Analyses

Case No.	Description	Factor of Safety
6	710 m reservoir, high rainfall	1.00
5	710 m reservoir, low rainfall	1.10
4	650 m reservoir, high rainfall	1.08
3	650 m reservoir, low rainfall	1.18
2	No reservoir, high rainfall	1.12
1	No reservoir, low rainfall	1.21

would remain marginally stable (factor of safety of 1.10) during periods of high reservoir levels up to 710 m and low rainfall. Marginal slope stability (factor of safety of 1.08) would also develop if heavy rainfalls occurred at reservoir elevations near 650 m. The movements of October 1960, corresponding to a factor of safety of 1.0, may have developed because of the abnormally heavy rainfalls during this period. It is probable that the groundwater levels in October 1960 were above the levels considered as "high" groundwater levels in these computations.

The "high" groundwater levels were derived, in part, from the observation that piezometer P2 was 90 m above reservoir elevation at about October 20, 1961. Rainfall amounts for 7, 15, 30 and 45 days before October 20, 1961, were 59, 205, 208 and 246 mm, respectively. Rainfall amounts for 7, 15, 30 and 45 days before October 31, 1960, were 109, 170, 495 and 697 mm, respectively. The rainfall preceding October 31, 1960, was much heavier than the rainfall preceding October 21, 1961, when the P2 piezometer was operational and yielded data that were used in establishing the "low" and "high" groundwater elevations for these analyses.

If the groundwater elevations were adjusted to take into account the heavier rainfall in October 1960, the factor of safety of 1.08 would more appropriately be reduced to near 1.0. The factor of safety of 1.18 for the 650 m reservoir elevation and low rainfall (see Table 11) is indicative of the stable conditions that were observed in January 1962 when the reservoir was raised through el. 650 m and no movement was observed. Marginal slope stability (factor of safety of 1.12) was estimated for periods of heavy rainfall even without the presence of the reservoir. Over periods of several hundreds of years, periods of rainfall were likely in which raised piezometric levels were high enough to reduce the factor of safety from 1.12 to 1.0, making the slope, with no reservoir, unstable.

The factors of safety presented in Table 11 indicate that rainfall significantly influenced the stability of the Vaiont slopes for any reservoir level. The movement, or lack of, at any level of the reservoir was greatly influenced by the intensity of rain over the preceding 15 to 30 days.

The other process, discussed by Müller, in which it was inferred that new movements only occurred when the reservoir was raised to new elevations exceeding previous reservoir elevations, would appear to be a result of making interpretations from movement and reservoir data without attaching the importance to rainfall that calculations here would suggest.[4,20]

The schematic diagram in Figure 42 shows the three-dimensional nature of the blocks selected for the calculation of forces acting in the upstream (easterly) direction. These blocks are designated as Block I, Block I + II and Block I + II + III. In Appendix D of Hendron and Patton, the factor of safety of Block I (Figure 42a) is shown to be 1.07 for $\phi = 12°$, $\beta = 40°$, $\phi_R = 36°$ and high rainfall with reservoir elevation at 710 m.[18] The factor of safety of Block I + II is 1.31 (Figure 42b) and the factor of safety of Block I + II + III is 1.00 (Figure 42c). These three-dimensional analyses, which consider the shear forces between sections caused by the upstream dip of the strata, account for the fact that the entire slide came down at one time. This finding was not apparent from the two-dimensional analyses that yielded calculated factors of safety for Sections 2, 5 and 10A that were quite different

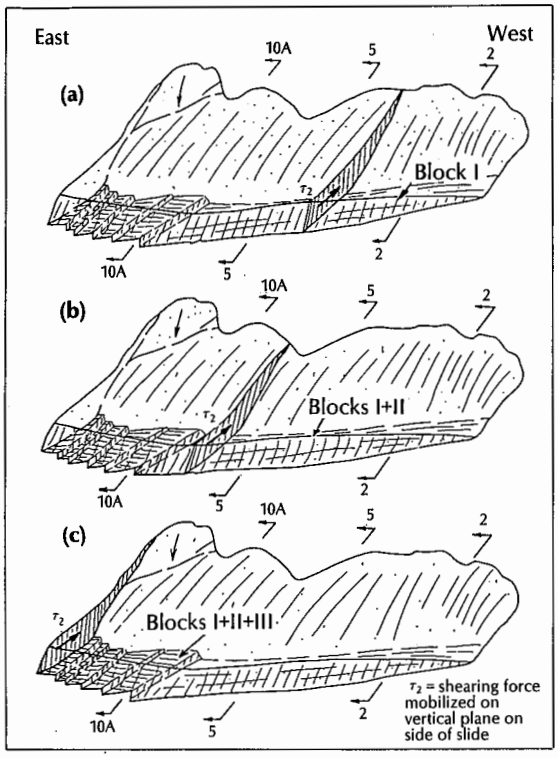


FIGURE 42. A depiction of the three blocks used in the three-dimensional stability analyses.

for each section.

The three-dimensional calculations also account for the fact that if the strength on the bedding planes were as low as 12°, the slide mass could have been stable, but not greatly above a factor of safety of 1.0, before the reservoir was built. However, this condition required large resistance along the eastern boundary of the slide. The epicenters of small tremors reported near the eastern boundary of the slide during the history of movement agree with the conclusion that significant resistance was developed at this boundary.

Conclusions

The 1963 Vaiont Slide was a reactivation of an old slide. Although the age of the old slide is not known, it probably occurred in post-glacial times, but before the period of recorded history of the Vaiont Valley. The evidence for an old slide includes: talus infilling a reoccurring crack at the headscarp where breccias occur with a variety of physical characteristics, the basal rupture plane and remnants of a previous slide mass or masses on the north side of the valley. Elements of surface morphology are also indicative of an old slide. These elements include deranged drainage, enclosed depressions, bulging slopes and other related alignments and patterns evident in the 1960 airphotos.

The slide mass moved on one or more clay layers that were continuous over large areas of the surface of sliding both east and west of the Massalezza Ditch. Multiple clay interbeds occur in the Malm and Lower Cretaceous stratigraphic units and were observed at many locations within the slide. Clays occur on the slide surface, below the slide surface and also form the matrix of the lower portions of the slide mass. Thick clay fragments and layers are abundant in the debris. Clay interbeds were found outside the slide area in stratigraphic positions corresponding to the surface of sliding of the 1963 slide. The field evidence for the presence of clay along the surface of sliding is compelling because of the number of locations where clays were noted on the failure surface and because of the details of the geology at these locations. Clays of predominantly calcium montmorillonite or a closely related clay mineral occur on the failure surface in many more locations than those cited in this study.

The lower portion of the failure plane, which is commonly seen as a near horizontal "seat" in cross sections of the Vaiont Slide, actually dips to the east (upstream) about 9° to 22°. This upstream dip is very significant in the stability analyses and is well documented by the geologic mapping of Giudici and Semenza before the slide and by drillholes made after the slide.[7]

The eastern boundary of the slide appears to have been formed by one or more lateral faults. Such a lateral fault is shown on the geologic maps of Rossi and Semenza.[26,32]

The great majority of the slide both east and west of the central Massalezza Ditch moved as a unit. The evidence for this movement is the surface morphology of the slide and the geologic features of the area mapped before and after the slide by Rossi and Semenza. A secondary slide movement

formed an area called the Eastern Lobe. This movement was presumably triggered by the loss of toe support caused by the movement of the main slide. The resulting unstable mass overran a large area on the uphill side of the eastern part of the slide. Therefore, an analysis of the main slide is not appreciably affected by omitting a consideration of the secondary slide. On this point, the authors differ with others who have suggested that the portion of the slide east of the Massalezza Ditch was fundamentally different from the portion to the west. As the main slide came to rest, differential movements developed within it as a result of differences in the geometry of the valley in the toe areas and differences in the momentum of various sections of the slide mass.

A significant area of pronounced karstic and/or combined karstic and glaciated terrain exists above the slide near the top of Mt. Toc. Evidence of minor and incipient karstic terrain is found just above the slide and on its western boundary. The bedding in those areas also dips towards the slide at angles of 13° to 45° or more. Solution features were observed at three areas immediately below the main surface of sliding. Undoubtedly, more solution features existed. This evidence strongly suggests that the conditions were present to enable the transmission of high water pressures developed due to infiltration from precipitation or snowmelt on the mountain above. These high water pressures could therefore develop along the surface of sliding.

High groundwater piezometric pressures with respect to the reservoir levels were measured in piezometer P2 in the vicinity of (probably just above) the failure surface. These measurements were taken prior to the slide and apparently before sufficient slide movement occurred to damage the piezometer. This water pressure fluctuated both with changes in the reservoir level and with rainfall. Initially, the piezometer level in P2 was 90 m above reservoir level. This level represents a water pressure difference that was probably lower than the real difference because the piezometer tip was not well sealed. Also, this 90 m difference was observed in a period of low to moderate rainfall and could have been higher in periods of high rainfall.

The lower permeability of the clay layers and the higher permeability of the intervening limestones and cherts must have combined to significantly increase the hydraulic conductivity along the bedding relative to that across the bedding. This effect results in a near classic case of an inclined multiple-layer artesian aquifer system at and below the surface of sliding. Such a system would be expected to produce the high piezometric levels observed at P2.

The values of the drained residual angle of shearing resistance of the clays measured in the laboratory varied from 5° to 16°, with most values of the clay-rich layers ranging from 6° to 10°. These values are consistent with the Atterberg limits of the clay sampled from a large number of areas throughout the slide and from the formation located outside the slide area. To account for irregularities along the clay layers and a limited number of rock-to-rock surfaces of contact, an average value of the residual angle of shearing resistance of about 12° would appear to be reasonable and consistent with laboratory test results.

Three-dimensional analyses were required due to the magnitude of the upstream inclination of the clay layers that form the slide base. These analyses reveal that a significant proportion, approximately 40 percent, of the total shearing resistance acting on the slide mass was supplied by near-vertical faces that formed the eastern boundary of the slide. This particular slide is especially sensitive to this three-dimensional effect because the clay layers along the base have a very low strength and the eastern boundary has a higher strength.

The history of slide movements, the record of reservoir levels, the shape of the failure surfaces and the assumed distribution of pore water pressures and water levels used in this study are consistent with the following shear strength values:

- residual angle of shear resistance (ϕ_r) on basal planes $\sim 12°$
- angle of internal shearing resistance (β) acting between slices on the slide $\sim 40°$

- angle of frictional shearing resistance acting along the eastern surface of the slide $\phi_R \sim 36°$

Only small variations in the parameters above appear to be possible for the results of the analyses to yield factors of safety consistent with the four periods of movement and the intervening periods of relative stability.

The 1963 slide occurred because of the combined effects of a rising reservoir and increases in piezometric levels as a result of rainfall. The reduction in the factor of safety caused by reservoir filling alone is calculated to be approximately 12 percent. The reduction in the factor of safety due only to a variation in rainfall and snowmelt is calculated to range from 10 to 18 percent.

Plots of cumulative precipitation against reservoir levels just prior to periods of movement have resulted in a well-defined "failure" envelope. This envelope indicates those combinations of reservoir and precipitation levels that yield a pore pressure distribution that would cause significant slide movement. The results of this correlation explain why the slide was observed to be stable at a given reservoir level and yet at a later date was unstable at the same reservoir level. The results of this correlation indicate that "prewetting" of the slide debris was not a significant factor in the slide behavior.

An extrapolation of the failure envelope enables an estimate to be made of:

- the rainfall that would cause failure without a reservoir
- the reservoir level that would cause failure with little or no rainfall on the Mt. Toc slopes

The cumulative 30-day rainfall that would cause failure without a reservoir would be about 700 mm. Since a monthly rainfall of almost 500 mm was recorded in the four-year period of record, it seems likely that the 700 mm rainfall has been exceeded during the post-glacial life of the slope. Therefore, significant movements must have occurred without a reservoir. The reservoir level that would cause failure without rainfall would be about 710 to 720 m. This reservoir failure level may be compared to the full supply level of the Vaiont Reservoir that was to have been 722.5 m. Therefore, had the reservoir been filled to its design level, the slide might have moved without any significant preceding rainfall. The results of the stability analyses are consistent with the conclusions that can be drawn from the precipitation against reservoir level correlations and the available movement record of the slide. A review of the accumulated evidence suggests that the slide could have been stabilized by drainage.

Casual studies of important precedent case histories, such as the Vaiont Slide, should not be accepted by the geological and geotechnical professions, especially when used for comparison purposes for proposed projects. Back-analyses and speculations on slide causes should not be made without a reasonably valid geologic, hydrogeologic and historic reconstruction of the significant events into a model. Because of the great diversity in geologic and hydrogeologic environments among projects, it is difficult, and perhaps misleading, to attempt to set rules for analyses and field exploration programs that would cover all landslide studies.

Any damsite investigation should include a detailed study of the proposed reservoir slopes. If old slides or areas susceptible to sliding are identified, a detailed evaluation of their relative stability under reservoir conditions should be required. The lesson afforded by Vaiont need not be relearned by another generation. However, it should not be a foregone conclusion that reservoir slopes will always be less stable with increased reservoir levels.

The analyses and evidence compiled indicate that the history of the sliding and the final collapse of the slope can be examined in quantitative terms. Conventional methods of analyses by limit-equilibrium techniques appear to be reliable if the input data are consistent with the geologic and hydrogeologic controls. The greatest gaps in the data accumulated on the Vaiont Slide involve the lack of substantive water pressure data and reliable movement records along the failure plane. Fluid pressure measurements taken

from piezometers installed at multiple levels within and below the slide would have provided the essential data for correlation with slide movements and reservoir levels. Reliable measurements of the depth of the failure plane and the magnitude of displacements along it would have helped to confirm the depth and size of the slide mass and would have brought more reliability to the correlation studies.

In hindsight, it appears significant that an early (1960-61) diagnosis of the kinematics of the lower part of the slide as similar to that of some glaciers (having zero horizontal velocity at the base increasing to a maximum at the glacier's surface) led those involved to divert their attention away from the field exploration required to locate the failure plane, as well as away from instrumentation and analytical efforts. Subsurface borehole deformation measurements would have shown the error in this hypothesis.

ACKNOWLEDGEMENTS — *As the Second Casagrande Lecture, this article was originally presented to the Boston Society of Civil Engineers Section/ASCE on April 18, 1985. The principal support for this study came from a report prepared under a research contract with the US Army Waterways Experiment Station (WES).*[18] *The authors wish to acknowledge the interest and support of Drs. Don C. Banks and Paul Fisher; and Walter C. Sherman and Drs. George Kiersch and Paul Hadala for their constructive comments on the initial draft of the WES report. The authors also wish to thank Professors Donald L. Anderson and Edoardo Semenza, as well as Semenza's associate Dr. Daniele Rossi, for their substantial contributions; Dr. Gabriel Fernandez and H. Rodney Smith for their assistance in field work and office studies; Dr. Camillo Linari of ENEL for providing pertinent documents; and the British Columbia Hydro and Power Authority for the use of results from the authors' earlier studies on the Vaiont Slide.*

ALFRED J. HENDRON, JR., *has been a Professor of Civil Engineering at the University of Illinois since 1965. He has been active in research in the areas of slope stability, construction blasting, tunnel lining design and soil dynamics. He received the ASCE Huber Research Prize in 1972. Dr. Hendron's areas of teaching have included rock mechanics, earth dams, soil dynamics, case studies and foundation engineering. He is an active consultant to government agencies and engineering firms in the U.S. and abroad on dams, tunnels, slopes and other geotechnical activities related to hydrolectric and nuclear powerplant projects. He was elected to the National Academy of Engineering in 1983.*

FRANKLIN D. PATTON *is a consulting engineering geologist in North Vancouver, BC, Canada. He received his B.Sc. in civil engineering from the University of Alberta, and his M.Sc. in civil engineering and Ph.D. in geology from the University of Illinois. He has taught engineering geology at the University of Wisconsin-Madison; the University of Illinois-Urbana; and the University of British Columbia-Vancouver. His engineering geology practice includes work on dams, landslides, tunnels and foundations. He is also the founder and president of Westbay Instruments Ltd., which designs and manufactures groundwater monitoring equipment.*

REFERENCES

1. Gruner, E.C., "Vigilance Over Reservoirs," *Water and Water Engineering*, September 1969, pp. 369-373.

2. Broili, L., "New Knowledge on the Geomorphology of the Vaiont Slide Slip Surfaces," *Rock Mechanics and Engr. Geol.*, Vol. V, No. 1, 1967, pp. 38-88.

3. Müller, L., "Discussion on Question 32," *Proc. 9th Int. Congress Large Dams*, Istanbul, Turkey, Vol. VI, 1967, pp. 124-134.

4. Müller, L., "New Considerations on the Vaiont Slide," *Rock Mechanics and Engr. Geol.*, Vol. 6, No. 1, 1968, pp. 1-91.

5. Trollope, D.H., "An Approximate Design Method for Slopes in Strain-Softening Materials," in *Design Methods in Rock Mechanics*, C. Fairhurst and S.L. Couch, eds., Proc. 16th Symposium on Rock Mechanics, Univ. Minnesota, Minneapolis, 1977, pp. 45-51.

6. Chowdhury, R., "Analysis of the Vaiont Slide — New Approach," *Rock Mechanics*, Vol. 11, pp. 29-38.

7. Giudici, F., and Semenza, E., *Studio geologico del serbatoio dei Vajont*, unpublished report, 1960, Part A: 21 pp. text, Part B: 68 photos with discussions 42 pp.

8. Carloni, G.C., and Mazzanti, R., "Aspetti geomorfologici della frana del Vaiont," *Revista Geografica Italiana*, Vol. 71, No. 3, September 1964, pp. 201-231 (with summary in English).

9. Carloni, G.C., and Mazzanti, R., "Rilevamento geologico della frana del Vaiont," *Annali del Museo Geologico di*

Bologna, Ser. 2, Vol. 32, Fasc. 1, 1964, pp. 105-138.

10. Kiersch, G.A., "Vaiont Reservoir Disaster," *Civil Engineering,* March 1964, pp. 32-39.

11. Selli, R., and Trevisan, L., "Caratteri e interpretazione della Frana del Vaiont," in *La Frana Del Vaiont,* Annali del Museo Geologico di Bologna, Ser. 2, Vol. 32, Fas. 1, 1964, pp. 8-104.

12. Semenza, E., "Sintesi degli studi geologici sulla frana del Vajont dal 1959 al 1964," *Memorie del Museo Tridentino di Scienze Naturali,* A. XXIX-XXX 1966-67, Trento, Vol. 16, No. 1, 1965, pp. 1-52. (See note Reference 18.)

13. Skempton, A.W., "Bedding-plane Slip, Residual Strength and the Vaiont Landslide," letter to the editor in *Geotechnique,* Vol. 16, No. 1, 1966, pp. 82-84.

14. Kenney, T.C., "Stability of the Vajont Valley Slope," discussion of a paper by L. Muller in *Rock Mechanics and Engr. Geol.,* Vol. 5, No. 5, 1967, pp. 10-16.

15. Kenney, T.C., "The Influence of Mineral Composition on the Residual Strength of Natural Soils," *Proc. Geotechnical Conference,* Norwegian Geotechnical Institute, Oslo, Vol. 1, 1967, pp. 123-129.

16. Nonveiller, E., "Discussion of Paper by V. Mencl on Mechanics of Landslides with Noncircular Surfaces with Special Reference to the Vaiont Slide," *Geotechnique,* Vol. 17, No. 2, 1967, pp. 170-171.

17. Nonveiller, E., "Shear Strength of Bedded and Jointed Rock as Determined from the Zalesina and Vajont Slides," *Proc. Geotechnical Conference,* Oslo, Vol. 1, 1967, pp. 289-294.

18. Hendron, A.J., Jr., and Patton, F.D., *The Vaiont Slide: A Geotechnical Analysis Based on New Geologic Observations of the Failure Surface, Technical Report GL-85-5, Vols. 1 & 2,* US Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS, 1985, 104 pp. Includes, as Appendix G, a translation of E. Semenza, 1965, 48 pp., cited as Reference 12 here.

19. Ajemian, R., "There the Dam Stood, Proud and Beautiful," *Life [Magazine],* Vol. 55, No. 12, 1963, pp. 30-41.

20. Müller, L., "The Rock Slide in the Vajont Valley," *Rock Mechanics and Engr. Geol.,* Vol 2., 1964, pp. 148-212.

21. Weber, E., "Die Katastrophe von Vaiont in Oberitalien," *Wasser-und Energiewirtschaft,* Vol. 56, No. 2/3, 1964, pp. 61-69.

22. Weiss, V.E.H., "Vajont — Geologische Betrachastungen zur Felsgleitung in den Stausee," *Steirische Beitrage zur Hydrogeologie,* No. 15/16, 1963/64, pp. 11-36.

23. Jaeger, C., "The Vaiont Rockslide, Part I," *Water Power,* March 1965, pp. 110-111.

24. Gervasoni, A., *Il Vajont e le responsabilità dei manager,* Bramante Editrice, Milan, 1969, 166 pp.

25. Martinis, B., *Stratigrafia della Valle del T. Vajont,* unpublished AGIP report, 1964, 30 pp.

26. Rossi, D., and Semenza, E., *Carte geologiche del versante settentrionale del Monte Toc e zone limitrofe, prima e dopo il fenomeno di scivolamento del 9 ottobre 1963,* Instituto di Geologia dell'Università di Ferrara, 1965, 2 maps.

27. Loriga, C.B., and Mantovani, M.G., "Le biofacies del Cretacico della Valle del Vaiont (Belluno)," *Riv. It. Paleont.,* Milan, Vol. 71, No. 4, 1965, pp. 1225-1248, tav. 110-114.

28. Loriga, C.B., and Mantovani, M.G., "Microbiostratigrafia della serie affiorante nella massa scivolata dal Mt. Toc (Vaiont) il 9 ottobre 1963 e alcune osservazioni su Foraminiferi, Radiolari, Calcisfere e Nannoconus," *Museo Tridentino di Scienze Naturali,* Tento, Sez. A, Vol. 47, No. 2, 1970, pp. 202-285.

29. Ciabatti, M., "La dinamica della frana del Vaiont," *Annali del Museo Geologico di Bologna,* Ser. 2, Vol. 32, 1964, pp. 139-154.

30. Boyer, R.A., "Etude géologique des environs de Longarone (Alpes Vénitiennes)," *Bull. Soc. Geol. France,* S. 4, Vol. 13, 1913, pp. 451-485.

31. Dal Piaz, G., *Relazione di massima su due sezioni della valle del Vajont prese in considerazione per progetti di barramento idraulico,* unpublished report for S.A.D.E., 1928, 4 pp.

32. Rossi, D., and Semenza, E., *Relazione definitiva sulle condizioni di stabilita della Costa Delle Ortiche (Vaiont),* unpublished, 1964, 18 pp.

33. Rossi, D., and Semenza, E., *Note illustrative delle carte geologiche e delle serie di profili del versante settentrionale del M. Toc, precedenti e posterori allo scivolamento del 9 ottobre 1963,* unpublished, 1965, 11 pp.

34. Rossi, D., and Semenza, E., "La bassa Valle del Vajont e lo scivolamento gravitativo del 9 ottobre 1963," in *Le Dolomiti, Vol. 2,* P. Leonardi, ed., Manfrini, Rovereto, 1967, pp. 937-944.

35. Semenza, E., "Zona del Duranno," in *Le Dolomiti, Vol. 2,* P. Leonardi, ed., Manfrini, Rovereto, 1967, pp. 945-953.

36. Leonardi, P., and Semenza, E., "Zona di Longarone," in *Le Dolomiti, Vol. 2,* P. Leonardi, ed., Manfrini, Rovereto, 1967, pp. 925-936.

37. Leonardi, P., ed., *Le Dolomiti, Vols. 1 & 2,* Manfrini, Rovereto, 1967, 1,571 pp.

38. Patton, F.D., and Deere, D.U., "Significant Geological Factors in Rock Slope Stability," in *Planning Open Pit Mines,* P.W.J. Van Reusburg, ed., Proc. Symposium on the Theoretical Background to the Planning of Open Pit Mines with Special Reference to Slope Stability, Johannesburg, 1970, pp. 143-151.

39. Frattini, M., Arredi, F., Boni, A., Fasso, C., and Scarsella, F., *Relazione sulle cause che hanno determinato la frana nel serbatoio del Vajont (9 Ottobre 1963),* Frattini Commission Report prepared for ENEL, 1964, 92 pp.

40. Patton, F.D., and Hendron, A.J., Jr., "General Report on 'Mass Movements'," Proc. 2nd Int. Congress, Int. Assoc. Engr. Geol. (AIEG), Sao Paolo, Vol. 2, 1974, pp. 1-57.

41. Müller, L., *Talsperre Vajont 15, Baugeologischer Bericht: "Die Felsgleitung im Bereich Toc,"* unpublished report to S.A.D.E., 1961.

42. Lo, K.Y., Lee, C.F., and Gelinas, P., "Alternative Interpretation of the Vaiont Slide," in *Stability of Rock Slopes,* E.J. Cording, ed., Proc. 13th Symposium on Rock Mechanics, Univ. of Illinois, Urbana, ASCE, New York, 1971, pp. 595-623.

43. The following laboratories were employed: Waterways

Experiment Station, Vicksburg, Mississippi; Dept. of Geology, University of Illinois, Urbana, Illinois; Thurber Consultants Ltd., Edmonton, Alberta, Canada; and Instituto Di Scienza e Tecnica Delle Construzoni Del Politecnico de Milano, Milan, Italy.

44. Olson, R.E., "Shearing Strengths of Kaolinite, Illite and Montmorillonite," *Journal Geotech. Engr. Div., ASCE,* Vol. 100, No. GT11, 1974, pp. 1215-1229.

45. Bozzi, C., Merla, G., Trevisan, L., Selli, R., and Viparelli, M., *Comissione di inchiesta sulla sciagura del Vaiont,* Relazione al Ministro Dei Lavori Pubblici (Bozzi Commission Report), 1964, 108 pp.

46. Müller, L., *Relazione Geomeccanica sulla Frana di Roccia del 9 Ottobre 1963,* Italian translation by ENEL-SADE of Muller's report of 2/9/64, 89 pp.

47. Mencl, V., "Mechanics of Landslides with Non-Circular Slip Surfaces with Special Reference to the Vaiont Slide," *Geotechnique,* Vol. 16, No. 4, pp. 329-337.

48. Khan, S.U., *Effect of Changes in Reservoir Level on the Stability of Natural Slopes,* M.Sc. thesis, Univ. of Alberta, Edmonton, Canada, 1971, 98 pp. plus appendices.

49. Jaeger, C., "The Vaiont Slide," in *Rock Mechanics and Engineering,* University Press, Cambridge, 1972, pp. 340-361.

50. Anderson, D.L., "Static Slope Stability Analysis Method Used for the Vaiont Slide Analyses," in Hendron and Patton, *The Vaiont Slide: A Geotechnical Analysis Based on New Geologic Observations of the Failure Surface, Vol. 2, Appendix B,* 1985, pp. B1-B5.

Streamflow Distribution in the Jones River Basin

For streamflow distribution estimates to be more accurate, they must take into account hydrogeological conditions as well as being based on surface-drainage area.

DAVID G. JOHNSON

STREAMFLOW DISTRIBUTION in a small drainage basin can be significantly influenced by geologic controls. In some instances, flow may not be proportional to the drainage area and the percent yield from subbasins may vary considerably. Water resources engineers, managers and planners should take note of the importance of geologic controls on streamflow distribution within river basins.

In 1983, a study was undertaken by the Massachusetts Office of the U.S. Geological Survey (USGS), in cooperation with the Massachusetts Division of Water Resources, to determine the distribution of streamflow, under base-flow conditions, in the Jones River basin in southeastern Massachusetts. The Jones River is located 25 miles southeast of Boston and 5 miles northwest of the town of Plymouth (see Figure 1). Its drainage area covers 20.0 square miles above the USGS gaging station at Kingston, including 4.09 square miles that drain into Silver Lake. During low-flow periods in late summer, the Silver Lake water level declines since its waters are diverted for municipal water supplies for the nearby towns of Brockton, Whitman and Hanson at the rate of about 16.4 cubic feet per second per day ((ft^3/s)/d). Therefore, the 4.09 square mile area above the lake's outlet was excluded from this study. The Jones River drains into Kingston Bay on Cape Cod Bay. Cranberry bogs and swampy areas dominate the landscape in this semi-rural area. In general, vegetation adjacent to the streams consists of low, dense shrubs and brush. Various pines predominate further away from the streams.

Discharge measurements were made at several locations on the Jones River and near the mouths of its major tributaries during low-flow periods in the summer of 1983 in order to determine streamflow distribution. This information was needed to estimate the need for conservation releases from a proposed water-supply reservoir in the Pine Brook subbasin. Stream yields were not expected to be proportional to surface drainage areas at low flows because of local variations in soil permeability, geology and differences between surface water and ground-

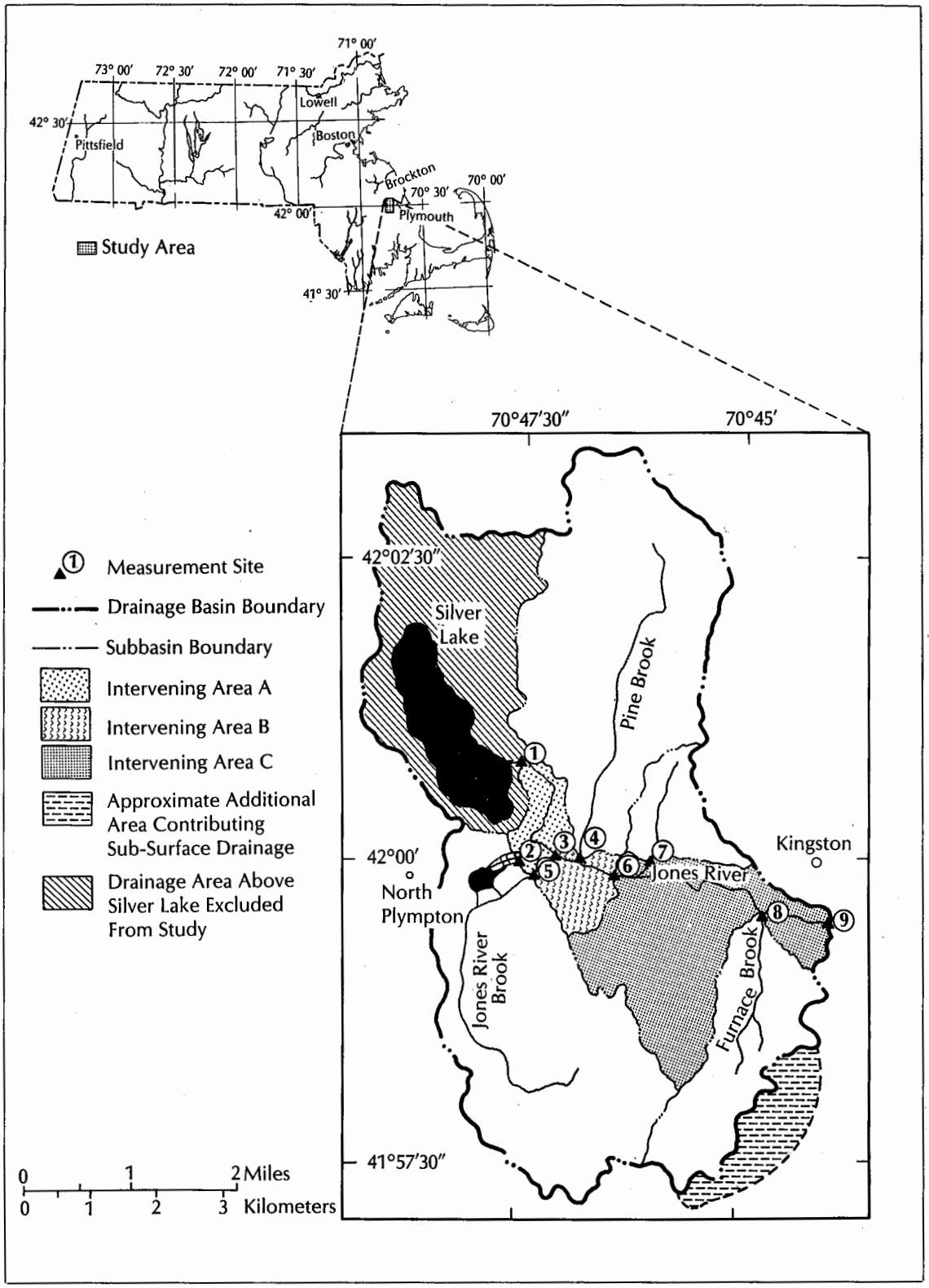


FIGURE 1. The location of the Jones River basin study area and measurement sites.

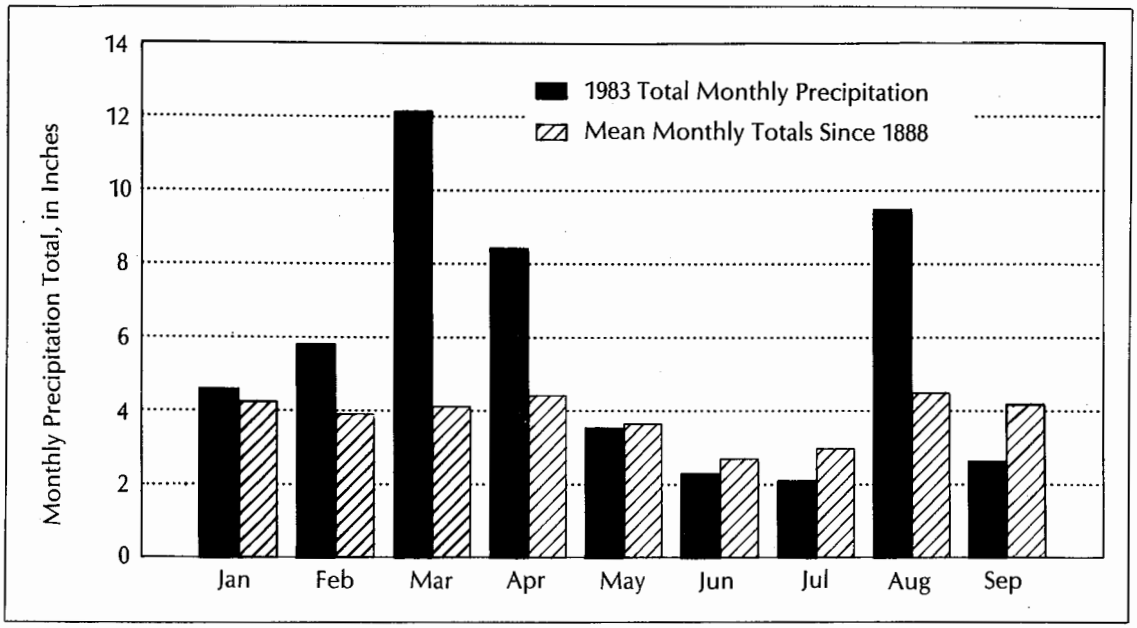


FIGURE 2. Comparison of 1983 monthly precipitation totals with mean monthly totals for the precipitation gage at Plymouth recorded over 95 years.

water divides.

Conditions

Sets of streamflow measurements were made August 2, 22 and September 12, 1983. For this basin, base flow was assumed to have been reached at least three days after a precipitation event. A very wet spring contributed to high groundwater levels throughout the summer. Monthly precipitation totals for the period of January to September 1983[1] were compared with mean monthly totals for the same months for the past 95 years[2] as recorded by a National Oceanic and Atmospheric Administration precipitation gage located four miles southeast of the study area in Plymouth (see Figure 2). Groundwater levels are illustrated in Figure 3, which compares the 1983 monthly groundwater level observations with the average observed monthly levels for the same months since October 1956 at the Plymouth 22 observation well six miles south of the study area.[3] This 42-foot deep well is in a water-table aquifer consisting of glacial outwash.

During this period, streamflow was higher than the historical average streamflow recorded at the U.S. Geological Survey gaging station on the Jones River at Kingston (see Figure 4).[4] Mean monthly discharges for 1983 were compared with the average mean monthly discharges recorded since August 1966. The mean monthly discharges for August and September 1983 were 21.4 and 17.8 cubic feet per second (ft^3/s), respectively; whereas the median recorded monthly discharges for these same months were 16.0 and 1.7 ft^3/s since August 1966.

Duration of flow at the U.S. Geological Survey gage on the Jones River at Kingston (the Kingston gage) was determined for the measurements that were made in order to show the percentage of time that these measured discharges were equaled or exceeded historically. Duration of flow can yield a perspective of how measured flow compares with the historical flow records without having to give consideration to seasonal variations in flow. Figure 5 shows a flow-duration curve of the Jones River at Kingston with the measured discharges of the study period plotted on the curve. This curve was based on the mean daily streamflow discharge record collected from August 1966 to September 1983. The duration of flow of the Jones River at the time of the streamflow measurements was 60,

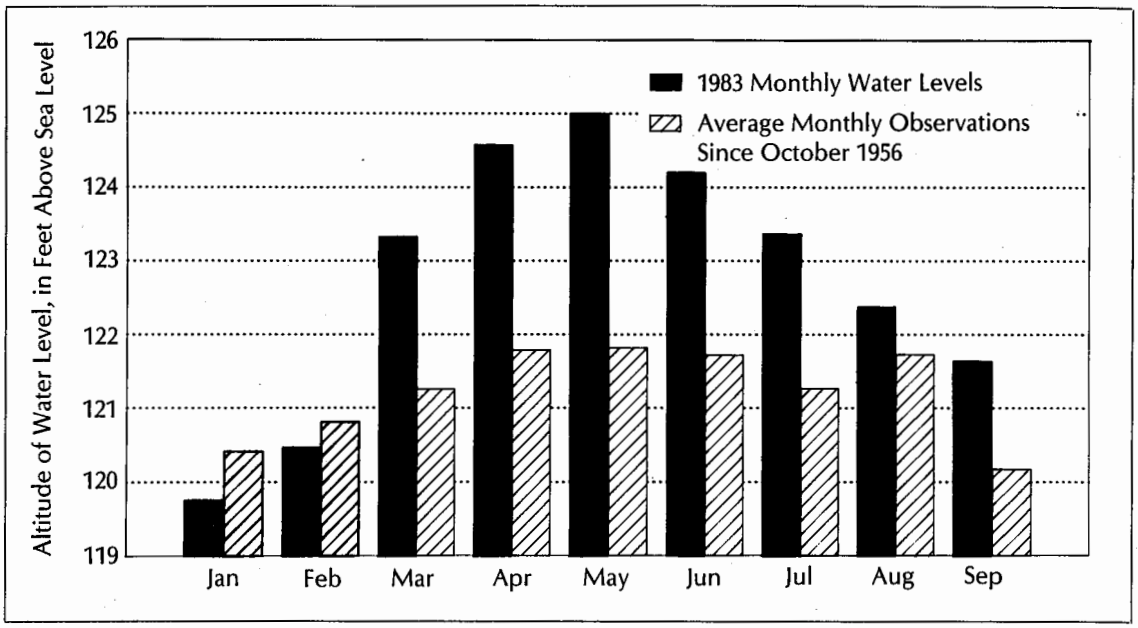


FIGURE 3. Comparison of 1983 monthly groundwater level observations with average monthly groundwater level observations since October 1956 for the Plymouth 22 observation well.

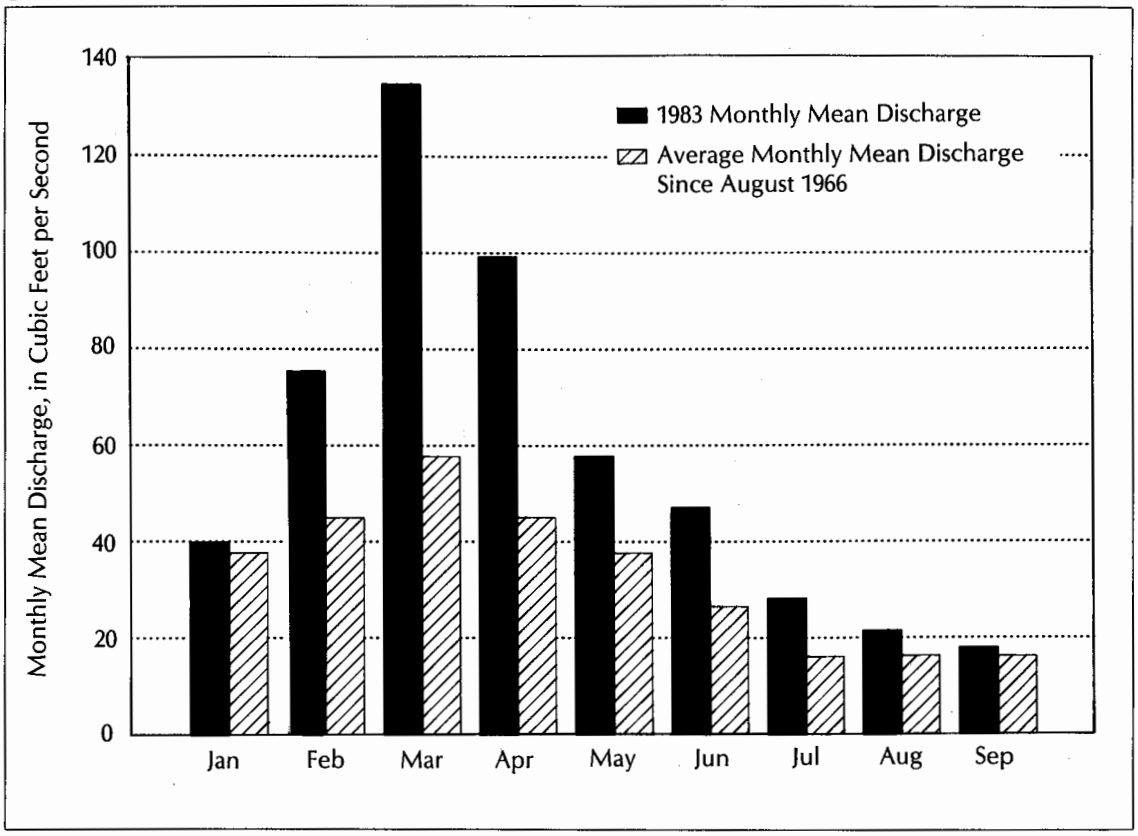


FIGURE 4. Comparison of 1983 monthly mean discharge with average monthly mean discharges since August 1966 for the Jones River near Kingston.

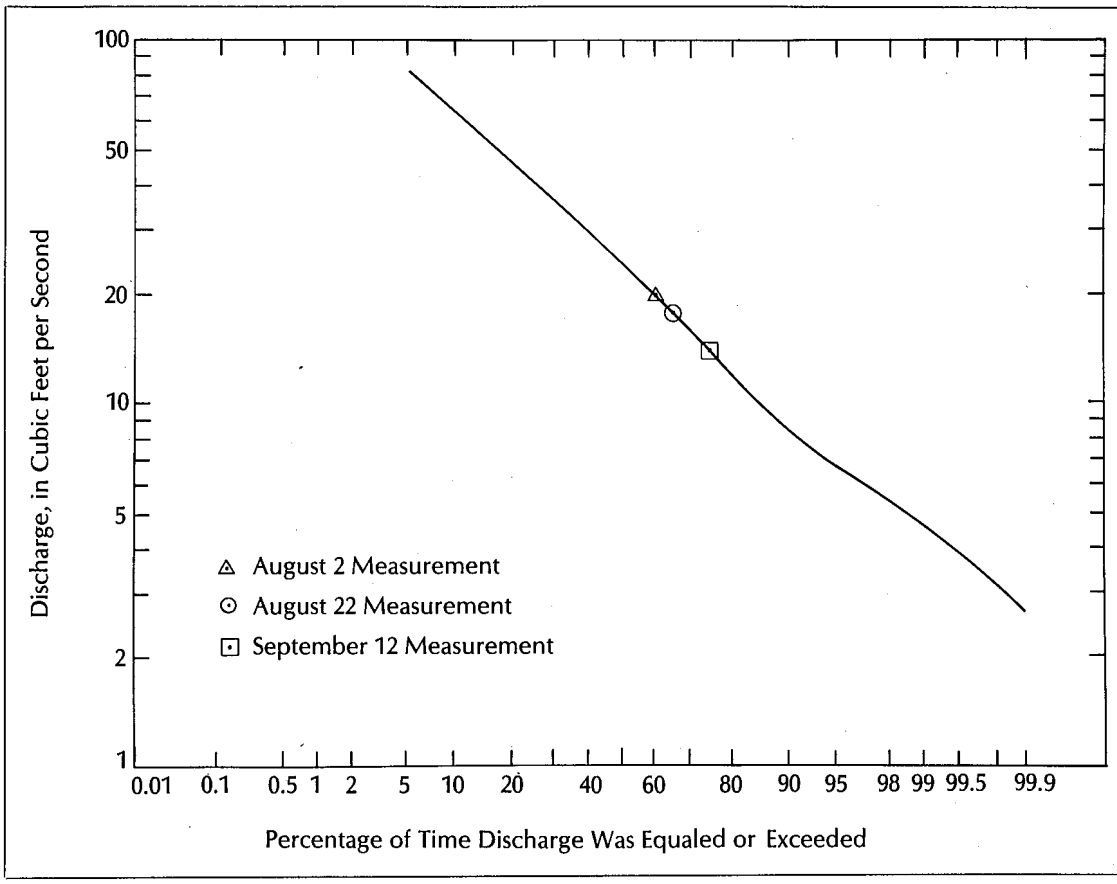


FIGURE 5. Flow duration curve for the Jones River near Kingston.

65 and 75 percent on August 2, 22 and September 12, respectively.

Surface runoff is generally low and infiltration capacity is high in the Jones River basin because of the low topographic relief and soil composition of the area. The topography is flat to gently sloping. The basin is composed of approximately 95 percent stratified drift that consists of permeable sand or gravel, underlain at shallow depth in some places by layers of less permeable silt and clay.[5] The permeability of surficial sand or gravel is 100 or more times greater than till or bedrock.[5] Evapotranspiration in the area averages about 22 inches per year, of which about 4.1 inches and 2.8 inches occur in August and September, respectively.[5]

Base-Flow Measurements

Streamflow measurements were made at nine sites along the Jones River and its tributaries on August 2, 22 and September 12, 1983. The locations of these measurement sites and their respective drainage areas are shown in Figure 1. Sites were selected at, or the near mouths of, tributary streams in order to assess their flow contributions above the Kingston gage. Additional sites at intermediate points along the main stem were selected to provide information on the intervening areas. Site selection was difficult because of the swampy nature of the area and the potential for irrigation pumpage across the drainage subdivides. Swampy conditions limited the number of potential measuring sites considerably because there were few reaches where the flow was confined to a single channel, or to a few channels in a small area, instead of flowing in multiple channels through wide, low, grassy areas. The cranberry bog area contained irrigation pipes that crossed drainage divides. These pipes, if in use, could divert water

TABLE 1

Summary of Base-Flow Measurements at Selected Sites

Site no.	Station no.	Stream name & location	Drainage area in square miles	August 2, 1983 Percentage of total drainage area	Discharge in cubic feet per second	Percentage of total discharge
1	011058561	Jones River below outlet of Silver Lake, near N. Plympton	0.09*	0.6	0.06	0.3
2	011058565	Jones River tributary near N. Plympton**	—	—	—	—
3	01105857	Jones River at N. Plympton	0.65*	4.1	0.64	3.2
4	01105859	Pine Brook near N. Plympton	4.72	29.7	1.25	6.3
5	01105861	Jones River Brook near Kingston	4.75	29.9	2.44	12.3
6	01105862	Jones River near Kingston	10.6*	66.7	3.35	16.8
7	01105863	Jones River tributary near Kingston	0.62	3.9	—	—
8	01105869	Furnace Brook at Kingston	2.23***	14.0	3.20	16.1
9	01105870	Jones River at Kingston	15.9*	100.0	19.9	100.0

*Excludes 4.09 mi.^{2} above the outlet of Silver Lake, from which flow is diverted for municipal water supply.
**All data included in Site 5 due to unknown pumping to and from cranberry bog pond.
***Topographic or surface-drainage area only, additional groundwater drainage area of about 0.9 mi.^{2} not included.

across the divides. Therefore, sites were selected to minimize such complications.

Measured discharges were compared to the drainage areas of the measurement sites, and to the discharge and drainage area for the Kingston gage (see Table 1). Similar comparisons were made for the intervening areas (see Table 2). The intervening areas are presented to demonstrate flow contributions from areas along the main stem. The intervening flow contributions were determined by subtracting the measurements for subdrainages upstream from those of the main stem sites; the results were the intervening contributions.

Because streamflow might be expected to vary directly with the size of the area from which it drains, flow from large and small areas was compared by converting the total flow from each area to flow per square mile of drainage area, with the resulting unit discharge expressed in cubic feet per second per square mile. Because streamflow is sustained primarily by discharge from groundwater storage during base flow, large unit discharge values represent large amounts of groundwater discharge and small values represent

	August 22, 1983			September 12, 1983		
Cubic feet per second per square mile	Discharge in cubic feet per second	Percentage of total discharge	Cubic feet per second per square mile	Discharge in cubic feet per second	Percentage of total discharge	Cubic feet per second per square mile
0.67	0.07	0.4	0.78	0.02	0.1	0.22
—	—	—	—	—	—	—
.98	0.62	3.5	0.95	0.52	3.7	0.80
0.26	1.83	10.3	0.39	0.70	5.0	0.15
0.51	4.63	26.0	0.97	2.10	15.0	0.44
0.32	6.03	33.9	0.56	4.41	31.5	0.42
—	0.04	0.2	0.06	0.01	0.1	0.02
1.43	3.30	18.5	1.48	2.65	18.9	1.19
1.25	17.8	100.0	1.13	14.0	100.0	0.88

small amounts of groundwater discharge.

Although Pine Brook (Site 4) drains 29.7 percent of the Jones River basin above the Kingston gage (Site 9), it contributed less than 10 percent of the discharge at base flow. Pine Brook contributed 0.26, 0.39 and 0.15 cubic feet per second per square mile ($(ft^3)/mi^2$) on August 2, 22 and September 12, respectively. On the same dates, the lower reach of the Jones River (intervening area C in Figure 1) contributed 5.45, 3.44 and 2.83 $(ft^3)/mi^2$, which represents 47 to 67 percent of the total flow from 15.4 percent of the total drainage area. Furnace Brook (Site 8), a tributary immediately above the lower reach of the river, contributed 1.19 to 1.48 $(ft^3)/mi^2$, or from 16 to 19 percent of the total flow from 14.0 percent of the total surface drainage area. On August 2, the unit discharge of Pine Brook was 0.26 $(ft^3)/mi^2$, and the reach of the Jones River above Site 9 (intervening area C) was 5.45 $(ft^3)/mi^2$, a difference of about 2,000 percent.

Interpretation of Measurements

The measurements were made at base flow, when streamflow is derived from basin stor-

TABLE 2

Summary of Calculated Base-Flow Contributions From Intervening Areas

Area	Location	Drainage area in square miles	Percentage of total drainage area	August 2, 1983 Discharge in cubic feet per second	Percentage of total discharge	Cubic feet per second per square mile
A	Jones River below Site 1, & above Site 3	0.56	3.5	0.58	2.9	1.04
B	Jones River below Sites 2, 3, 4, 5 & above Site 6	0.48	3.0	(-0.98)	—	—
C	Jones River below Sites 6, 7, 8 & above Site 9	2.45	15.4	13.35*	67.1	5.45

*Site 7 not measured.
( ) Indicate net losses.

age. Other factors affecting streamflow could be the diversion of water for irrigation or from pumpage of water-supply wells. There was no known diversion for irrigation of cranberry bogs at the times of measurements, although conversations with bog operators indicated irrigation may have taken place early in the morning, but more than three hours before the streamflow measurements were made. The water-supply test wells for the town of Brockton located near Pine Brook were not pumped at times of the measurements, but Kingston's supply wells, located near Furnace Brook, withdrew a total of 2.58, 0.67 and 1.90 (ft^3/s)/d on August 2, 22 and September 12, respectively. These flow figures, when added to the measured discharge from the Furnace Brook subbasin, indicated an even greater water yield from the lower part of the Jones River basin.

Groundwater-flow divides do not necessarily coincide with surface-drainage divides. For example, the high yield per square mile of Furnace Brook (see Table 1) indicated that some groundwater captured by Furnace Brook originated outside its topographic subbasin. Indeed, water levels in several kettle hole ponds showed that the southeasterly groundwater divide was a few thousand feet southeast of the surface-drainage divide, and in the intervening area of about 0.9 square miles (see Figure 1) the water table slopes northwest toward Furnace Brook. The upper (southern) reaches of Furnace Brook basin have thick, permeable stratified drift. However, near the mouth, the brook flows between two ridges of till, and the surficial sand is thin beneath its valley. This geometry suggests that there is restricted underflow in the lower reaches of the Furnace Brook basin. Therefore, nearly all basin yield would be forced into the channel.

The low values of unit discharge shown in the upper part of the basin and the high values in the lower part could be caused by groundwater underflow through the permea-

August 22, 1983			September 12, 1983		
Discharge in cubic feet per second	Percentage of total discharge	Cubic feet per second per square mile	Discharge in cubic feet per second	Percentage of total discharge	Cubic feet per second per square mile
0.55	3.1	0.98	0.50	3.6	0.89
(-1.05)	—	—	1.09	7.8	2.27
8.43	47.4	3.44	6.93	49.5	2.83

ble sandy glacial deposits of the basin. Precipitation falling on the upper part of the basin infiltrates the ground, drains down to the water table, and flows as groundwater, much of which discharges into the river in the reach immediately above the Kingston gage.

A quick approximation of the high-evapotranspiration area (swamps, lakes, cranberry bogs and streams) in each subbasin shows that swamps, lakes and low-lying flat areas make up a small fraction of the Furnace Brook basin and intervening area C. However, they constitute a much larger fraction of the subbasins in the upper reaches of the basin. The area of lakes and swamps is negatively correlated with low-flow indices.[6] The large areas of swamps, lakes and bogs in the subbasins in the upper reaches of the basin may be partly responsible for the below average yields of those subbasins.

These data represent base-flow conditions only. At high flows, distribution would be expected to be more nearly proportional to the surface-drainage areas. However, high-flow contributions in this basin may also be variable because of the high potential for groundwater recharge in sandy soils and the lack of coincidence between groundwater and surface-water divides. The nature of the surficial deposits in the basin provides the dominant control on the base-flow runoff characteristics of the streams in the basin. Other researchers have shown this relationship exists in Connecticut,[7,8] and other areas of southeastern Massachusetts.[9] The gentle slope of the Jones River flow-duration curve is representative of a stream in terrain with relatively high infiltration and storage capacity (see Figure 5). In this type of terrain, a significant amount of precipitation is held in storage and then gradually released, thereby providing a relatively high base flow.[7]

The effects of diversions — withdrawals from wells, groundwater underflow — and evapotranspiration on basin yield will not be as great during high flow as opposed to low

flow. Withdrawals and diversions probably do not increase in winter or spring; therefore, they comprise a lesser fraction of basin yield at high base flow. Since the downvalley groundwater gradient component does not significantly change seasonally, neither will underflow. Therefore, underflow constitutes a lesser fraction of basin yield at high flow than at low flow.

The variability of hydrogeologic conditions exhibited in the Jones River basin is probably fairly typical of other small basins in glaciated areas. Therefore, these conditions should be considered when attempting to determine streamflow distribution.

Conclusion

Discharge measurements made in the Jones River basin showed that at moderately low base flow, streamflow was not proportional to the surface-drainage area. The unit area yield from some subbasins varied as much as 2,000 percent during the measurements. Flow contributions were small in the upper reaches of the basin and large in the lower reaches. This disproportional distribution of streamflow can be largely attributed to the permeability of the sandy soils and groundwater underflow, partly to the incongruence of groundwater and surface-water drainage divides, and possibly to the predominance of high evapotranspiration areas (impoundments and wetlands) in the upper reaches.

Estimates of streamflow distribution, based solely on surface-drainage area without an analysis of area geology could lead to significant errors as had happened for the Jones River basin. Geology is of primary importance in determining streamflow distribution in a basin. Hydrologic measurements, such as those made in this study, can be considered very reliable for estimating flow distribution, especially in areas where the effect of drainage-basin characteristics are poorly known or understood.

ACKNOWLEDGEMENTS — *The author thanks Pine duBois, a Kingston Conservation Commission member, who was helpful in familiarizing him with the study area, and Dr. Michael H. Frimpter of the U.S. Geological Survey for his encouragement and assistance in preparing this article.*

DAVID G. JOHNSON *is an hydrologist with the U.S. Geological Survey Water Resources Division in Boston, Massachusetts. Since receiving his undergraduate degree at Pennsylvania State University in 1979, he has worked on a variety of groundwater and surface water related studies in Kansas and Massachusetts. He is currently project leader for a ground-penetrating radar demonstration study of water table mapping on Cape Cod and for a water resources inventory of the Westfield and Farmington River basins in western Massachusetts.*

REFERENCES

1. National Oceanic and Atmospheric Administration, *Climatological Data, Annual Summary, New England,* Asheville, NC, Vol. 94, No. 13, 1982.

2. National Oceanic and Atmospheric Administration, *Climatological Data, New England,* Asheville, NC, Vol. 95, Nos. 2-9, 1983.

3. U.S. Geological Survey, *Current Water Resources Condition in Central New England,* Boston, MA, monthly report, February-September 1983.

4. U.S. Geological Survey, *Water Resources Data for Massachusetts and Rhode Island, Water Year 1983,* USGS Water Data Report MA-RI-84-1, 1983.

5. Williams, J.R., and Tasker, G.D., *Water Resources of the Coastal Drainage Basins of Southeastern Massachusetts, Weir River, Hingham, to Jones River, Kingston: U.S. Geological Survey Hydrologic Investigations Atlas HA-504,* 1974.

6. Weiss, L.A., U.S. Geological Survey, Hartford, Connecticut, written communication based on unpublished data, 1984.

7. Thomas, M.P., "Effect of Glacial Geology upon the Time Distribution of Streamflow in Eastern and Southern Connecticut," in *Geological Survey Research 1966: U.S. Geological Survey Professional Paper 550-B,* 1966, p. B209-B212.

8. Cervoine, M.A., Melvin, R.L., and Cyr, K.A., "A Method for Estimating the 7-Day, 10-Year Low Flow of Streams in Connecticut," *Connecticut Water Resources Bulletin,* No. 34, 1982.

9. Tasker, G.D., "Estimating Low-Flow Characteristics of Streams in Southeastern Massachusetts from Maps of Ground-water Availability," in *Geological Survey Research 1972: U.S. Geological Survey Professional Paper 800-D,* 1972, p. D217-D220.

Lowell Waterpower System

Not many projects have had such wide-ranging effects as this engineering project had on the industrialization of America.

H. HOBART HOLLY

FEW ENGINEERING achievements have had greater impact on American life than the waterpower system developed at Lowell, Massachusetts. By making possible the generation of power at diverse locations rather than at a single damsite, the way was opened for the growth of the mill complex that became a feature of American life, especially in New England. Starting with Lowell, located some 20 miles northwest of Boston on the Merrimack River, mill towns grew into industrial cities. The advent of Lowell's waterpower system was a decisive factor in shifting America from a rural to urban society.

Demand for increased production capabilities caused the mill to develop from a small, relatively simple industrial plant to a large and complex manufactory in the 19th century. However, both the small plant and large manufactory were still dependent on waterpower for their operation. The invention of new types of production machinery helped spur this transformation, especially in the textile industry. The production machinery used in these new and larger mills required far more power than did the old small mills. In addition, there developed a need for power to be distributed beyond the practical limits of shafting and belts. Thus the civil engineer was called upon to make far-reaching contributions to the industrial revolution and to the resulting economic progress of this country.

Initially, increased power requirements were met by expanding on proven practices. Higher dams were built, and more and larger water wheels of the old types were used. The pioneer Slater Mill, built in 1793 at Pawtucket, Rhode Island, differed little basically in its power generation system from the small, earlier mills. Subsequent mills made incremental advances. At Slatersville, a few miles up the Blackstone River from the historic Slater Mill, there is still evidence of a somewhat more elaborate waterpower system. However, the engineering breakthrough that transformed American industrial life took place at Lowell, an accomplishment appropriately commemorated by the Lowell National Historical Park and the Lowell Heritage State Park.

Background

Two enterprises converged in 1821 that laid the framework for the development of the

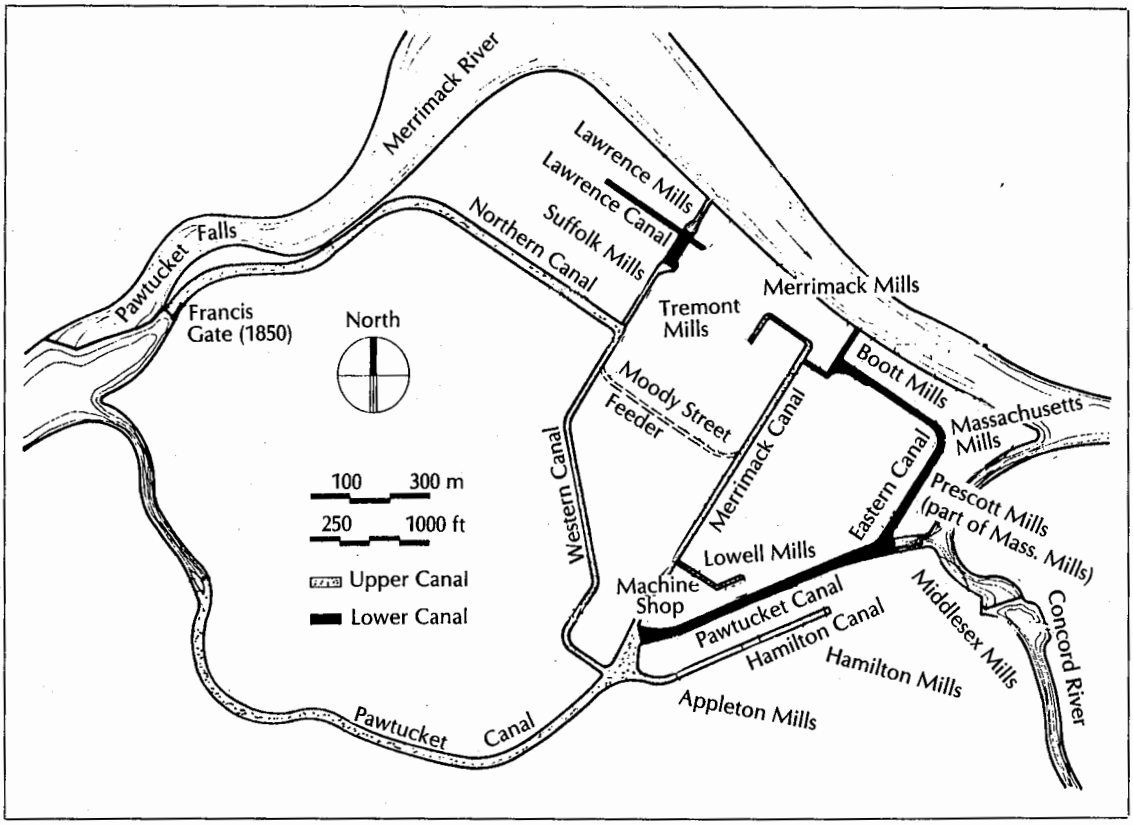


FIGURE 1. The Lowell canal system in 1848.

Lowell waterpower system. The Proprietors of Locks and Canals on the Merrimack River was incorporated in 1792 and it continues in existence today. This company acquired water rights at East Chelmsford, Massachusetts, and built a small transportation canal around the 30-foot Pawtucket Falls on the Merrimack River. This canal was completed in 1796 and a few small mills were built in the area. By 1821, however, the canal's condition had deteriorated and the company was financially pressed.

Meanwhile in 1813 a group of Boston merchants headed by Francis Cabot Lowell (1775-1817) started a textile mill experiment at Waltham, Massachusetts, on the Charles River. The Waltham mill was very successful, but limited waterpower on the Charles restricted mill expansion. In 1821, the group of Boston merchants, now headed by Nathan Appleton, acquired control of the Proprietors of the Locks and Canals, formed the Merrimack Manufacturing Company, and converted the Pawtucket Falls Canal into the main artery of a waterpower canal system. Kirk Boott became the company agent. In 1826, the town of East Chelmsford was renamed Lowell, in honor of Francis Cabot Lowell, and was incorporated as a city in 1836. By that year, under the direction of Kirk Boott and Paul Moody, a carefully planned canal system had been developed with twenty-six mills, two print shops and a number of machine shops operating on Merrimack River power.

Waterpower Engineering Developments

As advanced as the Lowell waterpower system in 1836 was, increasing demands for more power and continued expansion called for improved utilization of the water power resources available. The engineering developments resulting from these demands were:

- More efficient power generators

- Means to measure and control power water flows to assure proper power levels under all conditions to each of the many users
- Canal system improvements based on a more thorough knowledge of the hydraulics involved

In effecting these developments, Uriah Atherton Boyden and James Bicheno Francis made their great contributions to the civil engineering profession and American industry.

Uriah Atherton Boyden (1804-1879), engineer and inventor, originally came from Massachusetts but gained his first technical experience with his brother Seth Boyden, a manufacturer and noted inventor, in New Jersey. Returning to New England, he worked on the first survey for the Boston & Providence Railroad, and with the eminent engineer Loammi Baldwin, Jr., on the naval drydock at Charlestown, Massachusetts. With little formal education, Boyden went on to work on various railroad construction projects before he became engineer for the Amoskeag Manufacturing Company in Manchester, New Hampshire, where he designed the hydraulic works.

In Lowell's early industrial development, breast wheels were used for power generation. High breast wheels measured up to thirty feet in diameter and total bucket lengths measured up to sixty feet. These wheels were an improvement over the old overshot wheels, but the efficiencies were only about 60 percent. About 1840 Boyden started designing horizontal wheels of the Fourneyron turbine type, eventually incorporating so many improvements that they became known as Boyden wheels. His first turbine installed at the Appleton cotton mills at Lowell in 1844 was rated at 75 horsepower and had a power efficiency of 78 percent, a dramatic improvement over breast wheels. In 1846, he designed three 190-horsepower turbines with a power efficiency of 82 percent for the Appleton mills. These turbines included such improvements as a suspended top bearing, and better-designed scroll penstock and diffuser. His turbines were based on the spiral approach, letting water enter the turbine at uniform velocity.

James Bicheno Francis (1815-1892) was born in England and engaged in railroad work there before coming to this country in 1833. He initially worked on railroad construction here under the noted engineer George W. Whistler. Francis was associated in this work with Julius W. Adams and James P. Kirkwood who both later became presidents of the American Society of Civil Engineers (ASCE). It was with Whistler that Francis came to work in Lowell as a draftsman in the machine shop of the Proprietors of Locks and Canals at the age of eighteen. In 1837, Whistler resigned his position with the company and the young Francis was appointed engineer, starting an association with that company that lasted for 55 years. In 1845, he was made agent, or general manager, of the company in addition to engineer. Francis was a founding member of the Boston Society of Civil Engineers in 1848, and served as its president in 1874. He also served as president of ASCE in 1881 and was made an honorary member.

In 1846, Francis ran tests on Boyden's turbines and concluded that the Lowell mills should switch from breast wheels to turbines. The following year he designed his own turbine and installed it in the Pawtucket Gatehouse. He designed the gatehouse with special testing chambers and other features associated with scientific experimentation, making it one of our nation's first industrial research laboratories. The inward-flow type turbine measured almost nine feet in diameter and was installed in a vertical setting within a granite ashlar cylindrical wheel pit. His experiments on that turbine lead to an improved turbine with curved guides and buckets that was installed in the Boott Mills in 1849. This turbine made significant design contributions in the development of the modern, mixed-flow reaction turbine.

With far more potential waterpower in the Merrimack River at Lowell than was then being used for developed mill sites, the challenges were to design a canal system that could deliver adequate water to many sites, develop waterflow controls to assure equal distribution under all conditions, and develop

FIGURE 2. The dedication of the Lowell Waterpower System. From left to right are: Richard W. Karn, President of ASCE; Rodney Plourde, President of BSCES; and the author.

means to monitor flows for control and billing. Francis conducted 92 water flows experiments at the Pawtucket Gatehouse in 1851. Published in 1855 as *The Lowell Hydraulic Experiments*, the book remains an engineering classic for the completeness and accuracy of its test program as well as its landmark achievements in the hydraulics of open channels. Employing the hook-gauge invented by Boyden in his experiments, Francis used weirs as measuring devices and determined the formulas for their use. He also devised improved means of measuring flows in natural streams and channels.

As a result of the water flow data obtained from his experiments and using the means of measurement and control that were developed, far more efficient use could be made of the waterpower resources. Leasing and renting of waterpower became possible on a precise basis, and the distribution of water for power could be formulated on power requirements. The canal system at Lowell was subsequently designed by Francis to operate on a more advanced engineering basis. New canals were constructed and others modified by Francis. The Lowell canal system became a complex machine with many gates and other control and measuring devices. Francis' data, measurement means and design considerations were quickly applied to other locations, permitting the expansion of industrial mill centers. One particular feature of the canal system, Francis Gate, was constructed in 1850 on one of the new canals. After reviewing records of former floods, Francis had erected a gate to shut off flow to the canal. Dubbed "Francis' Folly" at the time, the gate saved Lowell from severe flood damage two years later and again in 1936, a tribute to his engineering genius and foresight.

Landmark Dedications

On July 1, 1985, at the Lowell National Historic Park, the American Society of Civil Engineers (ASCE) dedicated the Lowell Waterpower System as a National Historic Civil Engineering Landmark, and the American Society of Mechanical Engineers (ASME) dedicated the Pawtucket Gatehouse Hydraulic Turbine as a National Historic Mechanical Engineering Landmark.

President Rodney P. Plourde of BSCES presided at the joint ceremony. Brief remarks were given on the historical significance of the two new landmarks. ASCE President Richard W. Karn spoke on the ASCE history and heritage program, noting that Lowell was selected along with the Panama Canal and the Hoover Dam as landmarks the preceding fall. He presented a plaque commemorating the site to James Gutensohn, Commissioner of the Massachusetts Dept. of Environmental Management. The ASME plaque was presented by George Kotnick, past ASME President, to Marshall Field, Chairman of the Proprietors of Locks and Canals, Boott Mills and Boott Hydropower, owners of the system since its inception. Recognizing their historical importance, both sites are included in the Lowell National Historical Park by the National Park Service and the Massachusetts Dept. of Environmental Management.

ACKNOWLEDGEMENTS — *Figure 1 was drawn by Mark M. Howland and published in* ***The Lowell Canal System,*** *The Historic American Engineering Record, by Patrick M. Malone, 1976.*

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

BSCES: History & Heritage

Few professional societies in this nation have had a continued and rich history as the Boston Society of Civil Engineers. Nor have many societies had so many members whose contributions to our nation's growth have been so far-reaching.

GIAN S. LOMBARDO

THE LATE EIGHTEENTH century saw the great growth and spread of the industrial revolution in England. Giant strides in industrial activity necessitated the development of additional roads, canals and railways, as well as the improvement of rivers and harbors. It was during this period of initial industrial expansion that the profession of civil engineering was firmly established to meet these needs. Many men, most of them self-educated and self-made craftsmen and mechanics with exceptional talents, endeavored to realize these improvements. From among them two British engineers, John Smeaton (1724-1792) and Thomas Telford (1757-1834), made invaluable technical and social contributions toward the profession of civil engineering.

Initially a maker of scientific instruments, Smeaton was possibly the first to call himself a civil engineer. His talent and ability drew him into new fields and he was selected to work on the construction of the Eddystone Lighthouse. Subsequently, he worked, or consulted, on a wide range of road, bridge and canal construction projects. In 1771, those men in the new field of civil engineering created an informal organization to share common interests. Over the next 21 years, some 65 persons — engineers, scientists, craftsmen and mechanics — met to discuss their activities. By 1792 conflicts arose that put a halt to these meetings. However, there was sufficient momentum to renew the "society," but Smeaton died before meetings recommenced. In the following year a new organization of similar composition was formed and named the Smeatonian Society of Civil Engineers.

In 1817, a group of young engineers laid the foundation for starting a new organization that would meet more specific needs in the furthering of professional engineering knowledge. At a meeting on January 2, 1818, the group adopted rules that eventually became, with minor changes, the basis of the Institution of Civil Engineers. This new society met for two years without any substantial growth in membership. In an effort to spur member-

ship, the group altered its rules to include the election of a president "whose extensive practice as a civil engineer has gained him first-rate celebrity; and that a respectful communication be made to Thomas Telford, esquire, civil engineer, to patronize the Institution by taking upon himself the office of President."

Of humble origins, Telford was at first apprenticed to a stone mason. He mastered that craft and soon became involved in building projects, becoming county surveyor of Salop. Both in England and abroad, he became involved in bridge, canal, road and harbor construction. Telford was a member of the earlier Smeatonian Society, but had not been associated with the new society until being invited to serve as its president. Under his direction the new society grew rapidly and was incorporated on June 3, 1828. Several years later, Telford focused his attention more on the Institution of Civil Engineers and endowed it with a large bequest. His attentions assured this new society of its stature and longevity.

New engineering developments were also taking place in the United States. Two engineers in New England, Loammi Baldwin (1740-1807) and his son Loammi (1780-1838), made invaluable contributions to the profession and our nation's industrial growth. The elder Baldwin was the engineer for the Middlesex Canal between Boston and Lowell, Massachusetts. This pioneer project ushered in the canal era of American transportation. Utilizing locks and reservoirs, and with stream crossings and varied conditions to deal with, Baldwin established engineering practices that formed the standard for subsequent canal construction. The younger Baldwin was engineer of Dry Dock No. 1 at the Boston Naval Yard in Charlestown, Massachusetts. Finished in 1834, this dry dock served as the model for this type of construction. Both of these projects are National Historic Civil Engineering Landmarks. Other projects completed in the first half of the nineteenth century in New England that have been recognized as National Historic Civil Engineering Landmarks for their importance are:

Sewall's Bridge. Built in 1761 by Major Samuel Sewall, this bridge crosses the York River in York, Maine. It was this country's first pile trestle structure designed for general highway traffic. It was rebuilt on the original design in 1932.

Allendale Mills. Located on the Woonasquatucket River in North Providence, Rhode Island, the original mill building was a four-story stone rubble structure built in 1822 by Zachariah Allen. The building is one of the first applications of heavy timber set in mortar to increase fire resistance. The use of power looms for manufacturing broadcloth was pioneered here as well as the use of a rolling process to produce a gloss finish.

Granite Railway. The country's first incorporated railway that was operated as a transportation business. Built in 1826, this railway was used to move granite in Quincy, Massachusetts, from quarry to waterfront. Gridley Bryant was the engineer. The operating flexibility required for a common carrier resulted in innovations that are still used in railroad practice.

Cumberland & Oxford Canal. This canal from Portland to Sebago Lake in Maine was about 20 miles long. It had a rise of 265 ft., 27 locks and a small stream crossing on a 100-ft. wooden aqueduct. Designed by Holmes Hutchinson, engineer for the Erie Canal, it operated from 1830 to 1870.

Penobscot Boom. For about 100 years this complex of booms and crib-work piers along the Penobscot River in Maine played an important role in the area's prosperous lumbering industry. In 1835, the boom received and rafted over 82 million board feet of river-floated logs.

Canton Railroad Viaduct. Completed in 1834, this viaduct in Canton, Massachusetts, is still in use. One of the earliest surviving multi-arch stone railroad bridges in the country, it is 615 ft. in length. Engineers were G.W. Whistler and W.G. McNeill.

Lisbon Tunnel. Built in 1835-1837 by James Laurie as Chief Engineer of the Norwich and Worcester Railroad, it was the country's first true railroad tunnel. Located in Lisbon, Connecticut, it carried the railroad line through a hill of solid rock.

Babb's Bridge. Built in 1840 for $318, this bridge between Gorham and Windham, Maine, is the oldest covered bridge in Maine. A queenpost truss structure, it is still in operation.

Massachusetts Base Line. Authorized in 1830 and completed in 1841, this geodetic survey of Massachusetts from Hatfield to South Deerfield was the first such survey in any state. Simeon Borden performed the base line measurement with surveying equipment of his own invention.

Lowell Waterpower System. Due to the work of James B. Francis with waterflows, the canal system at Lowell, Massachusetts, was improved to provide efficient waterpower regulation. Francis, and U.A. Boyden before him, made design improvements to water turbines that resulted in higher power ratings. (See pages 141-144 for an article on this landmark.)

The Year 1848

Europe and North America prospered in the early nineteenth century under the impetus of the industrial revolution. In France, a group of engineers had finally succeeded in forming an organization to unite members of the civil engineering profession and to provide a forum for technical discussion. After many years of disagreement, primarily between government and private engineers, the Société des Ingénieurs Civils de France was formed on March 4, 1848.

In the United States, the nineteenth century saw great developments in highway, canal, tunnel and railway construction as well as water and sewer improvements based on the rediscovery of Roman techniques of bridge and aqueduct construction. Vast projects were conceived and carried out to tap the resources of this new land. Of the many distinguished events, developments and projects that occurred in the early nineteenth century, the year 1848 is notable for:

- The end of the Mexican War on February 2 that extended US territories from the Atlantic to Pacific Oceans, thus creating the need for a transcontinental railway.
- The discovery of gold on January 28 in California at Sutter's Mill, giving even more impetus for a transcontinental railway.
- The completion of the 1,200-ft., seventeen-arch Starrucca Viaduct in northeast Pennsylvania under the Scottish engineer James Kirkwood.
- Construction was begun on the first railroad across the Alps that would connect Vienna and Trieste.
- Thomas Telford completed his first tubular bridge (forerunner of the Britannia Bridge) 400 ft. across the Conway River.

Nearer to Boston, other events took place:

- Just eight years after opening, the Western Railroad of Massachusetts began double-tracking its line through the Berkshires.
- The Troy & Greenfield rail-line was in the process of organization, and was seeking to tunnel through Hoosac Mountain which it could not then undertake due to financing.
- Initial proposals for the destruction by surface blasting of the rock barriers at Hell's Gate, East River, New York, were made.
- The finishing of laying 51 miles of 5′ 6″ track from Portland, Maine, to Montreal.
- The New York & New Haven Railroad secured track rights into Manhattan, opening an all-rail route between Boston and New York.
- A new railroad was opened between Boston and Dedham, Massachusetts
- Fresh water was piped into Boston from Cochituate Pond about 20 miles away.

It was in this climate of expansion and innovation that the Boston Society of Civil Engineers — the first organization for the advancement of the civil engineering profession in the New World — was founded.

The Founding of the Society

The desire for civil engineers to unite for the advancement of the profession and for the improvement of its members was not confined to Europe. By 1848 the growth in the amount, and scope, of projects requiring civil engineers in the United States, and in Eastern Massachu-

setts in particular, had escalated. It seemed obvious to engineers in this area that they should form an organization that would further the practice of their profession. One of those engineers, Henry S. McKean, who attended the first meeting of what would become the Boston Society of Civil Engineers (BSCE), recounted the following about that meeting:

> "An informal meeting of gentlemen engaged principally in Civil Engineering, having been called by private notification, was held at the United States Hotel in Boston, on Wednesday evening, April 26, 1848, to consider the expediency of taking measures to form a Society, for social intercourse and professional improvement, to be composed of civil engineers and of gentlemen engaged in pursuits kindred to civil engineering. Of about twelve persons invited, there were present only Messrs. J.H. Blake, G.M. Dexter, E.S. Chesbrough, H.S. McKean and W.S. Whitwell.
>
> "The meeting was not formally organized. A general conversation took place on the subject of the proposed Society. A conviction that it was desirable to form one, and a willingness to cooperate for that object, was expressed by all present. The determination of the precise means to be used was deferred to a subsequent meeting which it was supposed would be more fully attended.
>
> "After a light supper the meeting adjourned to meet at the same place on Monday evening, May 8th."

Additional persons attended subsequent meetings at which the constitution and by-laws were adopted. The first regular meeting was held on July 3, 1848, when the Society elected the following officers: James F. Baldwin, President; George M. Dexter, Vice-President; John H. Blake, Secretary; William P. Parrott, Treasurer; and Joseph Bennett, Ellis S. Chesbrough, James Laurie, Samuel Nott and William S. Whitwell, Directors.

Thus, the nation's oldest continuing engineering society was established. In the 1830s there had been unsuccessful efforts to form an engineering society. In 1836, engineers associated with the Cincinnati & Charleston Railroad sought to establish the National Society of Civil Engineers. Three years later in Baltimore, forty engineers from eleven states proposed to form an American Society of Civil Engineers after a call from a meeting of engineers in Atlanta. Benjamin Latrobe, a railroad engineer, was elected president and a committee was formed to draw up a constitution. At a subsequent, poorly attended meeting in Philadelphia the constitution failed to receive approval and efforts to form this society ceased. Four years after the founding of the BSCE in 1848, the American Society of Civil Engineers and Architects was formed in New York. In 1869, the Civil Engineers Club of the Northwest (Western Society of Engineers) was established in Chicago.

Early Activities

The Boston Society of Civil Engineers secured a room for meetings in Joy's Building on Washington Street in Boston where one of the founding members of the society, Uriah A. Boyden, maintained an office. The society created a library committee to acquire technical and scientific books and periodicals for member use at Joy's Building.

A draft of the original by-laws stated that "every member be *required* to present a paper, or make a gift of a scientific book, map, plan or model each year." This by-law was subsequently amended to read that such activity would be "desirable means of promoting the objects of the Society." James Laurie presented the first paper to the Society on "Coal and Iron Trade of Great Britain and United States." Published in the *Mining Journal & American Railroad Gazette*, the editors commented that "judging from the paper of Mr. Laurie, we may look for valuable contributions from this Boston Society to the cause of science and general knowledge, and we rejoice in its existence among us."

A great majority of the papers presented to the Society in its first years of existence were on railroads and hydraulics since most members were engaged primarily in these fields. Such railroad-related papers included: "A Railroad to the Pacific," by P.P.F. Degrand,

Highlights of Founding Members' Careers

Samuel Ashburner (1816-1891) He spent most of his life in Boston, consulting in the railroad field. He had been division engineer on the construction of the Eastern Railroad, and chief engineer of the Hartford, Providence & Fishkill Railroad.

James F. Baldwin (1782-1862) Builder of the Boston & Lowell Railroad, he served as a commissioner for the construction of the Cochituate water supply. After its completion, his interests were mainly in connection with mills and industrial properties. He also served as a state senator.

Joseph Bennett (1814-1875) He worked first on railroads. He then worked on the Brooklyn Water Works and on surveys for increased power on the Merrimack River. He translated several technical books, was the Society's librarian and contributed articles to various publications.

John H. Blake (1808-1899) At first he had been in chemical practice. He was a partner with Darracott in building gas plants throughout New England. Later, he was active in street railway development and served as president of the Middlesex & Metropolitan Cos.

Simeon Borden (1798-1856) He was best known for his Trigonometrical Survey of Massachusetts. His services were widely sought as an expert witness on mechanical inventions.

Uriah A. Boyden (1804-1879) Inventor of the hook gauge and diffuser, he improved the Fourneyron turbine. He became financially independent through his inventions and devoted much of his life to scientific research.

Ellis S. Chesbrough (1813-1886) After 18 years in railroad work, he became chief engineer of the western end of the Cochituate water supply project, and later served as water commissioner and as Boston's first city engineer. In 1855, he went to Chicago, where for over 20 years he was in charge of sewerage, water supply and other public works.

John Childe (1802-1858) A graduate of West Point, he built much of the Western (B & A), Troy & Albany, Ohio Central and Mobile & Ohio railroads.

Marshall Conant (1801-1873) Editor of numerous publications on astronomy, he was a surveyor on the Boston water supply and later became principal of the Bridgewater Normal School.

Franklin Darracott (1820-1895) He was a partner with Blake in building gas plants throughout New England. He later went to New York City where he was vice president of the Nason Manufacturing Co., a firm dealing in valves, steam heating fittings and sanitary supplies.

William L. Dearborn (1812-1875) After locating and building railroads in various parts of New England, he became chief engineer of the Grand Junction Railroad. Later, he went to New York and was in charge of a number of important projects for the city's water supply.

George M. Dexter (1802-1872) He served as agent of the Boston & Lowell

continued

a Boston railroad broker; reports of railroad surveys; "Useful Formulae Adapted to Locating and Constructing Railroads," by Simeon Borden (later published in book form); and, reports on locomotive explosions. Other papers included: "Construction of Beacon Hill Reservoir" and "Mode Adopted for Carrying Water to South Boston," by W.S. Whitwell; "Contracts," by E.S. Chesbrough; and "Use of Lead for Service Pipes," by J.H. Blake. Simeon Borden exhibited his self-designed base-line measuring apparatus that was used for the

Railroad. He was interested in architecture and had much to do with development of buildings in Boston's old Scollay Square. Later, he was treasurer and then president of the Vermont Central Railroad.

Sereno B. Eaton (1823-1899) After working on the Old Colony Railroad, he moved to Ohio where he was engineer of the Keokuk, Fort des Moines & Minnesota Railroad and Hannibal & Naples Railroad; and other lines of the CB & Q system. During his later years, he was a contractor for heavy shovel work on railroad construction.

Robert H. Eddy (1812-1887) He first worked as engineer for the East Boston Land Co., developing much of that property. Later he became a patent solicitor, probably the first to establish that profession in this country.

Samuel M. Felton (1809-1889) He was chief engineer and superintendent of the Fitchburg Railroad, and president of the Philadelphia, Wilmington & Baltimore Railroad during the Civil War. He arranged for Abraham Lincoln's trip through Baltimore to take office as president. After the war, he became president of the Pennsylvania Steel Co.

James B. Francis (1815-1892) He became engineer of the Locks and Canals at Lowell at the age of 22, serving that company as engineer, agent and consulting engineer for over 50 years. He was responsible for turbine improvements, and for developing the waterpower system at Lowell. The published accounts of his water flows experiments, *Lowell Hydraulic Experiments,* in 1855 remains an engineering classic.

Charles H. Haswell (1809-1907) An eminent civil and marine engineer in New York City, he was the author of the *Engineers and Mechanics Handbook.*

Waldo Higginson (1814-1894) He served as agent of the Boston & Lowell Railroad. In his later years, he was founder and president of the Arkwright Mutual Fire Insurance Co.

Isaac Hinckley (1815-1888) He was superintendent of the Providence & Worcester Railroad, agent of the Merrimack Manufacturing Co., and on Felton's retirement became president of the Philadelphia, Wilmington & Baltimore Railroad.

Eben N. Horsford (1818-1893) He was a professor and founder of the Lawrence Scientific School at Harvard. He later founded the Rumford Chemical Works.

Josiah Hunt (1818-1874) He worked on Massachusetts railroads, becoming a superintendent in 1850. He moved west and became chief engineer of the Great Western (Illinois) Railroad, land agent of the Hannibal & St. Joseph Railroad, and director of many other lines. He was president of a bank and four-time mayor of Hannibal, MO.

Martin B. Inches (1820-1893) He was nephew of Boston mayor Martin Brimmer. He took part in railroad surveys and reported on water supplies in his early years.

Samuel F. Johnson (1821-1883) He was division engineer on the Fitchburg Railroad, engineer and superintendent of the Vermont & Massachusetts Railroad, chief engineer of the Chicago, St. Paul & Fond du Lac Railroad, and chief engineer of railroads in Texas. Returning to New England, he built parts of the Old Colony

trigonometrical survey of Massachusetts. Marshall Conant presented his "solar attachment" for a compass. With the new society well under way, it was incorporated on April 24, 1851, by George M. Dexter, Simeon Borden and William P. Parrott.

Early Civil Engineering Education & Career Opportunities

In the early nineteenth century very few civil engineers received formal training in their profession. Of all the early members of BSCE,

and the Portsmouth & Dover railroads.

James Laurie (1811-1875) He served on the construction of the Norwich & Worcester Railroad and on many railroad lines between New York and Boston. An expert on bridge construction, he built the bridge across the Connecticut River at Warehouse Point. He served as the first president of ASCE, and delivered the first papers before BSCE and ASCE.

Henry S. McKean (1810-1857) A teacher and librarian at one time, he then became resident engineer on the Boston water supply and assistant city engineer.

Samuel Nott (1815-1899) He worked on the Boston & Worcester Railroad. He was division engineer on the Eastern Railroad and later located and built many other railroads in New England. In 1855, he became superintendent of the Hartford, Providence & Fishkill Railroad.

George A. Parker (1822-1887) He was engineer on the Concord Railroad, chief engineer of the Sullivan Railroad, and superintendent and acting president of the Philadelphia, Wilmington & Baltimore Railroad. He built a $2 million bridge across the Susquehanna. Returning to New England, he built the Lancaster Railroad and served in the Massachusetts Legislature.

William P. Parrott (1810-1868) After engaging in the shipping industry, he worked as a partner with Nott on railroad surveys. He was later president of the Vermont & Canada Railroad and active in the development of Boston's waterfront, Back Bay and East Boston.

Thomas W. Pratt (1812-1875) He was involved in the construction of the Norwich & Worcester Railroad and other lines between New York and Boston. He developed the truss that bears his name and was inventor of various railroad mechanical improvements.

Theophilus E. Sickels (1822-1885) He served on the Croton Aqueduct and resident engineer under Chesbrough on the Cochituate Aqueduct. He then became chief engineer of the West Chester & Philadelphia and the Philadelphia & Baltimore Central railroads. Later he was chief engineer of the Hannibal & St. Joseph Railroad, chief engineer and general superintendent of the Union Pacific Railroad, and built the first bridge across the Missouri River at Omaha.

Lucian Tilton (1811-1877) He worked on Massachusetts railroads, becoming a superintendent in 1850. He was chief engineer and superintendent of the Toledo, Wabash & Western Railroad; president of the Great Western (Illinois) Railroad; director of the Illinois Central Railroad; and vice president of the North Chicago Railroad.

William S. Whitwell (1809-1899) He was chief engineer of the Boston distribution system of the Cochituate water supply and was for some years a partner with J.B. Henck. Later, he was treasurer and president of the Boston Water Power Co. and managing treasurer of the Boston & Roxbury Mill Dam Corp.

Thomas S. Williams (1812-1874) He was resident engineer on the Cochituate Aqueduct, superintendent of the Boston & Maine Railroad, and later a partner in Williams & Page, dealers in railroad supplies.

only Eben N. Horsford, an honorary member, had received a degree in civil engineering (from Rensselaer in 1838). Others received formal educations in related fields. William P. Parrott studied mathematics and natural science at Norwich University (1825); and Thomas W. Pratt attended Rensselaer. Many engineers were graduates of West Point where Sylvanus Thayer had pioneered technological education in this country. Among them were founding member John Childe, as well as G.W. Whistler, W.G. McNeill and W.H. Swift.

A number of engineers received their training by working in the offices of noted practicing engineers. The younger Loammi Baldwin was one such engineer who took on students. He had originally planned to become a lawyer, but his interest in natural science led him to engineering. He went to Europe to study public works and opened an office in Charlestown, Massachusetts, on his return. As Baldwin's reputation grew, young men came to him for training in physics, mathematics and surveying, and later served as his assistants. A number of BSCE founding members — Uriah Boyden, George Dexter, Robert Eddy, Samuel Felton, Waldo Higginson, Samuel Johnson, Henry McKean and George Parker — were trained in Baldwin's office. After his death, Samuel Felton took over the office and training activities.

Other founding members had only the equivalent of a high school education. These men learned their profession through practice and self-initiated study. James B. Francis and Ellis Chesbrough were two such men who went on to excel in their fields.

Opportunities abounded for these early engineers. From 1835-1855 railroad mileage increased in Massachusetts from 50 to 1,200 miles, and in New England from 50 to 3,500 miles. This explosion in railroad construction offered many opportunities and at least 28 of the 31 regular members had worked on railroads at some point. Other members worked on the water supply system for the growing city of Boston. Several engineers, James B. Francis and Uriah A. Boyden among them, were employed in the development of water power required by textile mills, especially along the Merrimack River north of Boston. Other engineers were involved in land and waterfront development, and road, bridge and gas works construction.

The growing need for engineers brought about a marked rise in educational institutions offering engineering courses and degrees. In 1847, the Lawrence Scientific School was established at Harvard, and the Sheffield Scientific School at Yale. Union College at Schenectady, New York, began offering engineering courses in 1845, as did Brown University at Providence, Rhode Island, in 1849. The Massachusetts Institute of Technology (MIT) was established in 1861 with BSCE members James Francis, William Parrott and Eben Horsford among the incorporators.

The Junior BSCE

The Boston water supply system was completed by 1850 and over the next few years the amount of railroad construction in New England diminished. However, the midwest was undergoing a period of rapid development, creating a demand for civil engineers. During the 1850s many Boston area engineers travelled west to take advantage of these new opportunities. Founding members John Childe, Sereno Eaton, Samuel Felton, Josiah Hunt, Samuel Johnson, John Laurie, George Parker, Theophilus Sickels and Lucian Tilton had become employed in railroad work outside of New England. Samuel Nott and Thomas Pratt were in Connecticut; and Joseph Bennett was in New York. Ellis Chesbrough left to oversee the Chicago sewer system. The Society continued to meet, but attendance was gradually dwindling. Since BSCE membership mainly consisted of engineers who had reached positions of responsibility within the profession, there was little encouragement given to younger men just entering the profession to become heavily involved in the Society's proceedings. The Society finally decided it was best to suspend activities temporarily. Samuel Ashburner and Waldo Higginson were given charge over the Society's property, and the Society's records and library were deposited with the Boston Athenaeum in 1861.

However, young engineers brought renewed life to the Society. During the 1860s a new generation of civil engineers arose in Boston. MIT had been founded in 1861 and classes started in 1865, with 1868 as the year of the first graduating class. One MIT graduate, Ernest W. Bowditch (1869), wrote to forty young engineers in the Boston area on May 24, 1873, about forming "a junior Engineers' Association to meet at certain times and places for the purpose of consulting and discussing whatever topics of interest may be suggested, connected with engineering."

Twenty-six persons met at MIT on May 30, 1873, in response to Bowditch's call. They

agreed to form a society with the suggested name of "Boston Engineers' Club." A draft of a constitution and by-laws were discussed at a second meeting. This initial constitution proposed the name "Boston Society of Engineers" for the group. The name provoked considerable discussion and Desmond FitzGerald finally proposed the name "Boston Society of Civil Engineers" which was accepted. Additional articles of the constitution were discussed, and just a week after the first meeting, the new society was formed with the following officers: Desmond FitzGerald, President; Henry Manley and Ernest Bowditch, Vice-Presidents; George Rice, Secretary; and, Robert Richards, Treasurer. Of the 62 signers of the constitution, at least 20 were recent graduates of MIT, and some signers were apparently enrolled as students there.

At the next two meetings, the constitution and by-laws were formalized, an agreement made with the Boston Public Library for the use of engineering books by society members, and arrangements made with MIT for the use of a room for meetings. Subsequent meetings were held regularly and papers were presented. The new society charged an executive committee on June 27, 1873, to explore actions necessary "to render the society legally capable of holding property and of having exclusive right to its name." This committee soon learned that another society incorporated in 1851 bore the same name. Thereupon, the society decided to look into a "union of the societies and possession of literary property of the corporated society."

Bowditch contacted about ten members of the older society who were still living around Boston. A warrant issued by George Morrill, Justice of the Peace, authorized members of the older society Thomas Pratt, Waldo Higginson and Franklin Darracott to call a legal meeting of the corporation. On April 27, 1874, original founding members James B. Francis, Waldo Higginson, Franklin Darracott, Thomas Pratt and Samuel Nott met and elected Francis to the office of president and Nott to clerk and secretary. The names of members of what was then called the Junior Society (BSCE of 1873) were proposed for membership in the older society, and Higginson and Pratt were instructed to reclaim property deposited with the Boston Athenaeum in 1851.

A meeting on June 8, 1874, saw the members of the Junior Society elected to membership in the older society, thus providing the BSCE with its current continuous link to the older society formed in 1848. Once the union had been accomplished, the older members retired from office and Thomas Doane was elected President, with Desmond FitzGerald, Vice-President; George Rice, Secretary; and Clemens Herschel, Treasurer.

Links With Other Societies

Just three weeks before the initial meeting of those interested in forming a civil engineering society, on April 5, 1848, a group of railroad superintendents met to form the New England Association of Railroad Superintendents. Original BSCE founding members Samuel Felton, Waldo Higginson, Josiah Hunt and Lucian Tilton attended. Other BSCE members to eventually join were William Parrott, Isaac Hinckley, Thomas Pratt, Samuel Johnson, Thomas Williams, George Dexter and Samuel Ashburner. Parrott was Secretary, Tilton served at one time as President, and Williams as Vice-President. The Association published a railway gazette that often included BSCE papers. The activities of the Society and the Association were closely related; later on the two organizations occupied adjacent rooms, sharing a library. With the development of the midwest and the slowdown in New England, rail construction members left the area, and the Association dissolved on October 1, 1857.

The American Society of Civil Engineers and Architects was formed on November 5, 1852, in the office of Alfred W. Craven, chief engineer of the New York City Croton Aqueduct. Twelve men were present, including James Laurie, a founding member of the original BSCE. He brought with him a constitution based on BSCE's and was elected President of that society, an office he held for 15 years. The American Society held regular meetings until March 2, 1855, when the society suspended activities. Again, like BSCE, engi-

neers were moving steadily west, creating an apparent void at home.

In 1867, due to Laurie's efforts, the American Society was revived. Its name was changed to the American Society of Civil Engineers (ASCE) the following year, and it has continued to flourish and advance the civil engineering profession ever since. Other original BSCE founding members who were members of ASCE at one time or another were Ellis Chesbrough, William Dearborn, George Dexter, Samuel Felton, James B. Francis, Charles Haswell, Waldo Higginson, Josiah Hunt, Martin Inches, George Parker, William Parrott, Thomas Pratt and Theophilus Sickels. In addition to Laurie, Chesbrough and Francis served as ASCE presidents during their careers.

The Northeastern Section of ASCE was formed in 1921, later becoming the Massachusetts Section. At that time, ASCE and BSCE formed a joint committee to explore forming an affiliation. This committee recommended affiliation and both societies approved this recommendation. However, the Affiliated Technical Societies of New England, later named Engineering Societies of New England (ESNE), was created in 1922. Sharing headquarters with ESNE at 715 Tremont Temple, BSCE found the affiliation with ASCE to be apparently unnecessary and it was mutually dissolved. The ESNE Journal publishes announcements of affiliated society meetings and matters of general interest to engineering societies in New England.

The Association of Engineering Societies was created in 1881 by BSCE, the Engineers' Club of St. Louis, Civil Engineers' Club of Cleveland and the Western Society of Engineers. The purpose of this Association was to jointly publish papers and proceedings of its participating societies. By 1900, the Association had eleven member societies, and in 1913 BSCE withdrew from the Association in order to publish its papers and proceedings independently.

Merger With ASCE

The Society has always had close ties with ASCE from the involvement of James Laurie in ASCE's founding to the 1921-1922 affiliation to joint meetings with the Massachusetts Section of ASCE in the 1960s. Having two prominent professional societies in one area could, and did, lead to scheduling conflicts and redundancy. Membership in both organizations was common. In 1957, a joint meeting of the ASCE Section and BSCE was held on the topic, "Why Two Civil Engineering Societies for Boston?" A committee report to BSCE in 1961 requested careful study of consolidation. Conflicts in meeting schedules between the two societies prompted the formation of more joint committees, and separate committees within each society, on relations with the other organization during the late 1960s. These interorganizational committees issued joint reports in 1970 and 1971 detailing recommendations for the consolidation of the two societies.

Merger with the Massachusetts Section of ASCE was viewed as a way to reduce the aura of friendly competition that had grown over the years, and to emphasize the aspects of cooperation. Such benefits of merger were:

- An increase in membership, reduced operating costs and increased meeting attendance.
- A lessening of the burden on persons holding key positions in both organizations, permitting them to fulfill their duties with less duplication of effort.
- Aiding the engineer in involvement in community, civic and political arenas through a single unified front.
- Attracting more young engineers who might not join BSCE because their residence in Boston might be temporary.
- Offering members the technical and professional strengths of both societies.

Upon a vote of the membership, BSCE agreed to absorb the Massachusetts Section of ASCE and to affiliate with ASCE. The Massachusetts corporate status and vested funds of the Society were retained, and under the affiliation the following changes were made:

- The Society changed its name to the Boston Society of Civil Engineers Section of the American Society of Civil Engineers

(BSCES), with its constitution and by-laws modified accordingly.

- BSCE members became members of the affiliated society without having to become members of the National ASCE.
- New, non-ASCE members would be designated junior affiliates without voting and office-holding privileges.

In April 1974, Thomas K. Liu became the first president of the Boston Society of Civil Engineers Section/ASCE. The following year the constitution and by-laws of BSCES were adopted.

Membership & Growth

The 1848 and 1875 constitution and by-laws provided for the grades of Member, Corresponding Member and Honorary Member. By 1890 the grade of Corresponding Member had been dropped and the grade of Associate added to provide for "persons interested in the objects of the Society and desirous of being connected with it." Joseph R. Worcester, in his 1909 presidential address, put great emphasis on making the Society more interesting to young engineers. Based on committee recommendation, the Society added the grade of Junior in 1910, and Student in 1932.

Society membership was 723 in 1910, and grew to 1,000 in 1928. It dropped to approximately 800 in 1948. Before association with ASCE in 1973, membership was at 1928 levels. Consolidation did indeed increase membership and in 1979 it was 2,000. Membership now is about 2,800.

Publications

Papers presented to the Society were bound and made available to the membership in the Society's library. One of the first BSCE publications was *Report of Proceedings — September, 1879 to June, 1881.*

The major reason for entering the Association of Engineering Societies in 1881 was to provide a means for jointly publishing papers and proceedings of the participating societies. From 1900 to 1906 the Association published a monthly publication that included meeting notices, and a special section entitled, "Bulletin of New Engineering Work." This bulletin gave way to an expanded monthly bulletin in 1906 that included all member society proceedings, with the exception of papers presented, and an advertising section.

In 1913, BSCE withdrew from the Association and established its own journal. Since the Society had been supplying nearly one-half of the Association's Journal, the Society felt that having its own journal would outweigh the loss of other society papers and the cost would be less by eliminating its share in the Monthly Bulletin. The first issue of the *BSCE Journal* appeared in January 1914. It contained papers presented before the Society, reports of professional committees, items of general interest and Society proceedings. The *BSCE Journal* was published ten times a year until 1934; quarterly from 1934 to 1981; and thereafter twice a year.

In 1983 the Society decided to make some changes in the focus and content of its Journal. The Society's Board of Government appointed a Journal Review Committee "to make a comprehensive evaluation of the Journal, taking into account all pertinent factors — financial, editorial, advertising, readership and production." As a result of that committee's recommendations, the following year the Board of Government endorsed "a major restructuring of the entire Journal effort...to obtain its permanent improvement." An enlarged and active Editorial Board was charged with undertaking intensive solicitation and review of papers as well as the revision and reorganization of the Journal's appearance, production, advertising and circulation. The first issue of *Civil Engineering Practice: Journal of BSCES,* which appeared earlier this year, represents the outcome of the evaluation begun in 1983.

Library

In 1848, a library committee was formed to acquire technical and scientific books and periodicals. Joseph Bennett took special interest in the library and was made official librarian in 1851. The library contained, in addition to books and periodicals, papers presented to the Society as well as plans, models and items of interest to members. In January 1853, the library and Society offices

were moved from Joy's Building to Tremont Row where the library was shared with the New England Association of Railroad Superintendents.

Upon deactivization of the original Society in 1861, the library was deposited with the Boston Athenaeum. When the Junior Society was inducted into the older society in 1874, there was some difficulty in reestablishing the library since some books had been catalogued and placed on the Boston Athenaeum Library shelves. Books that had not been incorporated into the Athenaeum Library were transferred back to the Society, and the Society was reimbursed for those that had been incorporated.

For about 25 years after 1874, older materials — particularly bound periodicals and departmental reports — had to be discarded to make room for new material due to the library's small quarters. By 1889 there were 600 bound volumes and 900 unbound volumes and pamphlets.

The library received a number of books of historical value in 1901 from the estate of Charles H. Swan. In 1906, Clemens Herschel, a past president, presented 70 books that were to form a nucleus of a special library. He continued to donate books throughout his life, and bequeathed some from his estate. Howard K. Parker in 1909 donated 300 volumes that showed civil engineering developments in the early nineteenth century. In 1916, the library received 1,100 books from the estate of Edmund K. Turner. In addition, many other members donated books over the years.

Securing the latest technical literature had always been a priority of the library. In 1922, books in electrical and mechanical fields were added and the list of periodicals broadened. By 1925 there were 10,500 bound volumes and 3,800 pamphlets.

During the 1960s and 1970s, restrictions on affordable space regrettably forced the gradual abandonment of the library, with the exception of copies of the Society's Journal and certain publications such as contributions to soil mechanics and lecture series notes. The library collection was distributed among local universities, and in May 1972 the bulk of the library collection was turned over to Northeastern University, with the condition that all Society members be granted Northeastern University Library privileges.

Technical Activities

In order to pay special attention to developments within specific disciplines within civil engineering, Albert F. Noyes, in his 1896 presidential address suggested the formation of groups within the Society for discussion of topics of particular interest in the various fields. This appeal was echoed in 1903 by President George A. Kimball. Later that year, 14 members petitioned for the establishment of "a section for consideration of the special subjects relating to sanitary engineering." Over the years additional sections were formed, or sections renamed, to reflect current technical emphasis (after affiliation with ASCE the designation "Section" was changed to "Group"):

Designers. Formed in 1920 and renamed *Structural* in 1947. As new sections were formed, its focus was increasingly devoted to structural engineering.

Northeastern University. Formed in 1922 to include students, graduates and faculty of Northeastern University. Disbanded in 1953.

Highway. First formed in 1924, renamed *Transportation* in 1946, with scope of activities broadened to include railways and airports.

Hydraulics. Formed in 1940.

Surveying & Mapping. Formed in 1940 and disbanded in 1964.

Construction. Formed in 1957.

Computer. Formed in 1969.

Geotechnical. Formed in 1969.

Environmental. Initially, this was the Sanitary section. It was renamed to Environmental in 1973.

Waterways, Port, Coastal & Ocean. Formed in 1983.

Engineering Management. Formed in 1984.

These technical groups hold from three to five technical meetings throughout the year. In addition, there are also from five to fifteen special meetings that include lecture series.

Awards, Prizes & Funds

In order to more fully recognize the contributions of its members to the profession, community and the Society, as well as to foster the development of students in civil engineering, the Society has administered various funded awards and prizes:

Desmond FitzGerald Award for promising students.

Howe-Walker Award to members of student chapters.

William P. Morse Award for students.

Lester Gaynor Award to an individual whose service as a part-time elected or appointed city official has been outstanding.

Clemens Herschel Award for a paper presented to the Society that has been useful and commendable.

Ralph W. Horne Award to a BSCE member whose unpaid service in municipal, state or federal elective or appointive post has been outstanding, or for philanthropic activity in the public interest.

Technical Group Awards for papers of excellence presented before technical group meetings.

President's Awards for members' contributions to, and efforts for, the Society.

Endowments to the Society also make possible:

Thomas R. Camp Lecture Series on outstanding recent developments or proposed or completed research in the sanitary engineering field.

Arthur Casagrande Lecture Series given by an eminent engineer with longstanding achievements in practice, teaching and/or research in geotechnical engineering.

John R. Freeman Fund particularly devoted to the encouragement of young engineers in various ways.

Joseph C. Lawler Lecture Series on the management of complex engineering projects.

Other Activities

Over the years, the Society has engaged in many other activities:

Code of Ethics. The Society developed and adopted, on December 18, 1912, one of the first engineering codes of ethics in the United States.

Employment & Welfare. In 1889 the Society directed the Secretary to maintain a list of members seeking employment, and established an employment bureau in 1910. This bureau was incorporated into the Affiliated Technical Societies of Boston in 1922 (now ESNE). In 1932, during the depression, the ESNE and Boston Society of Architects organized the Emergency Planning and Research Bureau, Inc., to render employment services. A Welfare Committee was also active during the depression and was concerned with matters of compensation and member welfare.

Boston Subsoils. In 1921 a Committee on Boston Subsoils was created and from 1923-1931 collected about 3,900 boring records. These records were published in the September 1931 *BSCE Journal.* Borings have been made periodically over the years, with results published in the Journal. The last such report was in 1985.

The Society has also had committees on rainfall and runoff, and floods in New England. In addition, the Society has drafted or revised building codes for state and municipal authorities.

ACKNOWLEDGEMENTS — *The greater portion of this paper is drawn from "The Boston Society of Civil Engineers and Its Founder Members," by John B. Babcock, in the* ***Journal of the Boston Society of Civil Engineers,*** *Vol. XXIII, No. 3, July 1936, pp. 151-180. Other sources include: "The Challenge of Change," by James P. Archibald, in the* ***Journal of the BSCE,*** *Vol. 60, No. 2, pp. 33-40; and, "A History of Progress," in the* ***Journal of the BSCE,*** *Vol. 65, No. 4, January 1979, pp. 113-133.*

GIAN S. LOMBARDO *is Editor of* ***Civil Engineering Practice.***

CIVIL ENGINEERING PRACTICE
BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE
THE ENGINEERING CENTER
236 HUNTINGTON AVE.
BOSTON, MA 02115-4701

Non-Profit Org.
U.S. Postage
PAID
Boston, MA
Permit No. 312