CIVIL ENGINEERING PRACTICE

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

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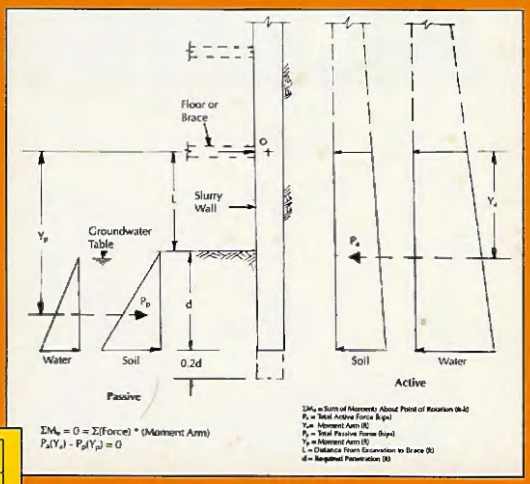
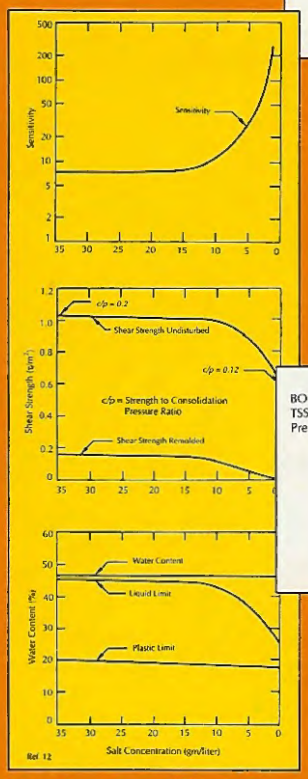
Slurry Wall Design



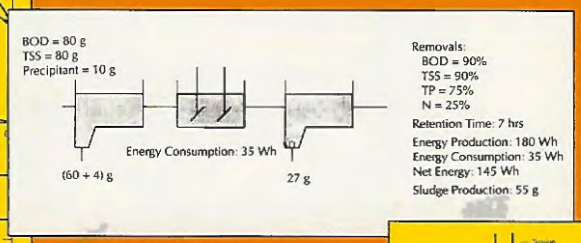
Understanding Soil Behavior



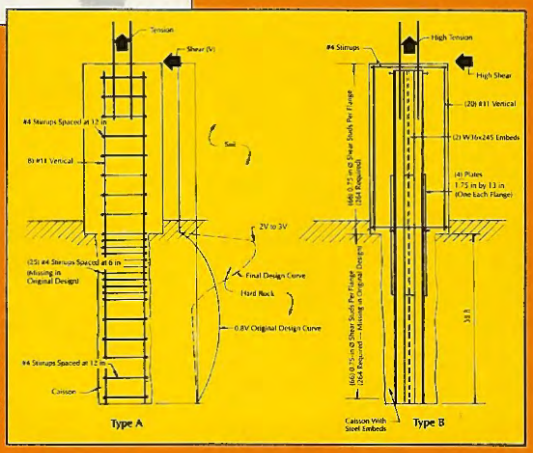
Also in This Issue:

- **The Effectiveness of Municipal Wastewater Treatment**

Chemically Enhanced Wastewater Treatment



Structural Engineering Peer Review



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Errata

In the Spring/Summer 1994 issue (Vol. 9, No. 1) — In Dominique Brocard's article, "The Scope of the Boston Harbor Project," on the first paragraph in the second column on page 5, a sentence should read: "The population of the area serviced by the Boston water supply then was about 1.5 million so the cost of the Quabbin project was roughly $300 per person (in 1993 dollars)." Also, Kevin Kirwin photographed the color picture on the front cover.

Understanding-Soil-Behavior Runs Through It

Understanding what soils are and the principles that govern their geotechnical properties are essential to sound geotechnical engineering practice.

JAMES K. MITCHELL

Geotechnical engineering is a continually evolving discipline, with shifting emphases and new challenges as needs and priorities change in the world around us. Although the focus of today's practice is on problems that are far different from those of 50 years ago, and the concerns of the 21st century can be expected to be much different from those of today, the key to their solution has depended, and will continue to depend, on the proper understanding of soil and rock properties and behavior when subjected to the many forces of nature as well as the activities of humankind.

Before examining specific considerations of some special and unique aspects of soil behavior, however, it is instructive to review very briefly how the geotechnical field has developed and how problems have changed over the past 50 years to illustrate the extensive breadth and wide diversity of problems in geotechnology and how new subdisciplines have emerged as new problems have arisen.

The Changing Focus of Geotechnical Engineering

The concerns, challenges and developments within geotechnical engineering over the past 50 years, as viewed within five- or 10-year periods, might look something like the following, which has been prepared with full realization that important topics may have been omitted and that there may be some disagreement over some of the selections and dates indicated:

1940–1950: Soil mechanics comes of age through the efforts of the early giants of the field such as Terzaghi, Casagrande, Peck, Taylor, and Skempton; each of whom built the mechanics on a carefully developed understanding of the properties and behavior of real soils.

1950–1960: Shear strength, bearing capacity, slope stability, soil structure and sensitivity, compacted clay, repeated loading, pavement design, soil stabilization, transient loading and soil dynamics.

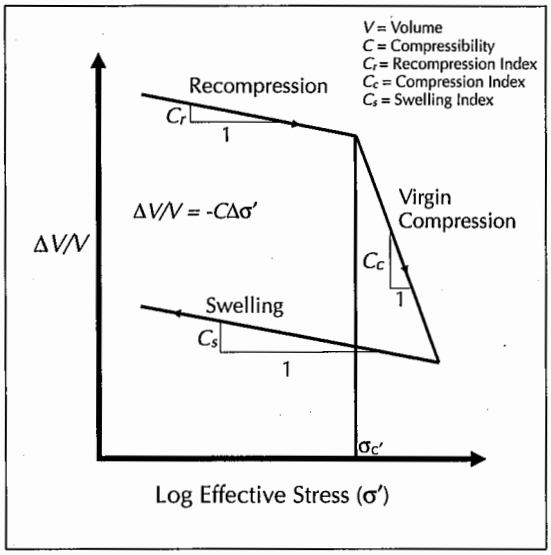


FIGURE 1. Idealized relationship for volume change (inert soil model).

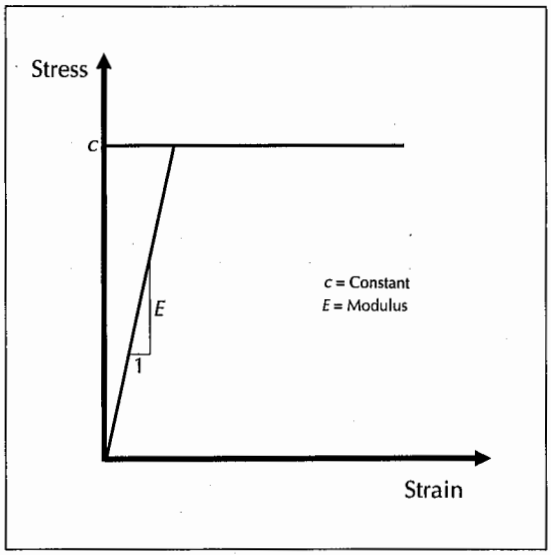


FIGURE 2. Idealized relationship for stress-strain (inert soil model).

1960–1965: Pore pressures and effective stress analysis, physico-chemical aspects of soil behavior, early development of rock mechanics, increased use of computer applications and introduction of the finite element method for the analysis of soil deformation and soil-structure interaction problems, Anchorage and Niigata earthquakes spawn earthquake geotechnical engineering.

1965–1970: Geotechnical engineering for offshore structures, nuclear power plants, lunar soil mechanics, soil-structure interaction.

1970–1975: Computer applications, soil dynamics, ground improvement.

1975–1980: In-situ measurement of soil properties, constitutive modeling, centrifuge testing, re-recognition of the importance of properties.

1980–1985: Earth reinforcement, geotechnical earthquake engineering, beginnings of geoenvironmental engineering, geosynthetics, ground improvement, risk and reliability.

1985–1990: Waste containment, landfills, site remediation, seismic risk mitigation, geosynthetics.

1990–1995: Geoenvironmental engineering, risk mitigation, infrastructure, properties for numerical analysis, land reclamation.

It seems that as new problems emerge (some of them unfortunately as a result of failures of one kind or another), as the need to treat and improve soils to make them suitable as foundation materials or for construction increases, and as we seek better and more efficient ways to predict future performance, attention keeps returning to basic questions of what soil is and why it responds the way it does.

Soils as Environmentally Sensitive Materials

If soils were inert and insensitive to their composition and the environment in which they exist, it is likely that designs, construction and expectations for future performance could be reduced to sets of rules, charts and graphs that could quickly transform what we do from a profession to a trade. Fortunately, or unfortunately, depending on one's perspective, soils are not inert materials, and their properties are very sensitive to and dependent on the environment (temperature, pressure, and chemical and biological regimes) in which they exist. Furthermore, the possible ranges and variations in particle size, composition, size distribution, void ratios, saturations, pressures, temperatures, and chemical and biological environments means that we are confronted with an almost infinite range of materials, each

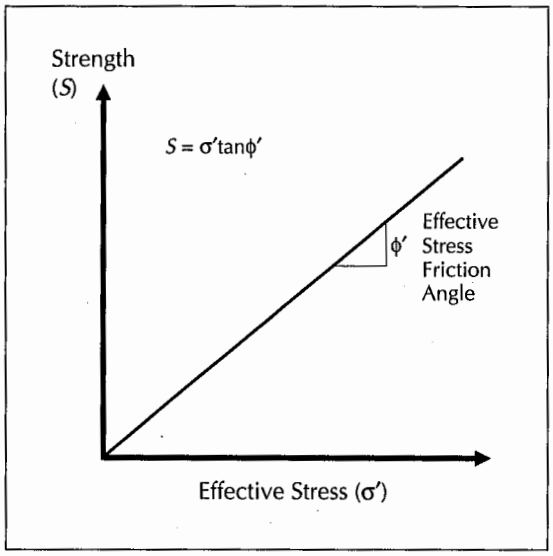


FIGURE 3. Idealized relationship for strength (inert soil model).

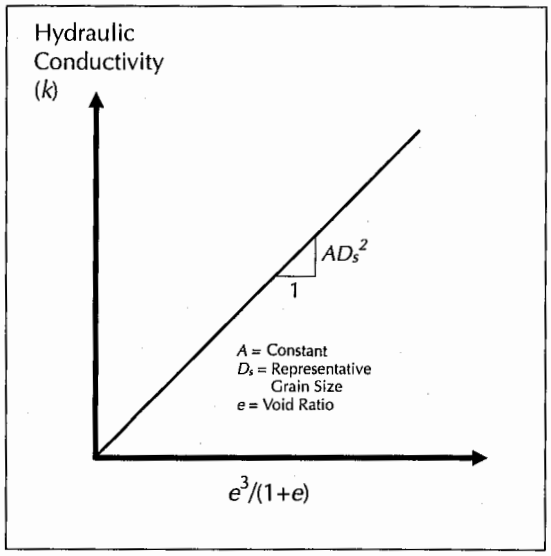


FIGURE 4. Idealized relationship for hydraulic conductivity (inert soil model).

with its own unique set of mechanical properties.

The following soil factors are of most importance in any project:

- Volume change characteristics;
- Stress-strain response;
- Strength, stress-strain-time properties;
- Conduction properties (hydraulic, thermal, chemical and electrical);
- Chemical and biological conditions; and,
- The variations of these properties with time and changes in temperature and pressure.

Any single property can usually vary over a wide range, depending on how it is measured and defined. Therefore, careful consideration of testing conditions and analytical modeling in relation to the particular problem or application of interest is also necessary.

Inert & Real Soil Models. If soil particles were inert and unaffected by physico-chemical interactions with the environment and other particles around them, then analyses and designs could be performed using rather simple time-independent relationships that describe properties over a wide range of conditions. For example, an idealized relationship between volume change and effective stress is shown in Figure 1, between deviatoric stress and strain in Figure 2, between effective stress and strength in Figure 3, and between hydraulic conductivity and void ratio in Figure 4. In these relationships the effective stress, σ', for saturated soils is defined by the difference between the total stress, σ, and the pore water pressure, u. To account for differences in grain size, grain shape and grain size distribution among different soils, families of curves of the type shown in Figures 1 through 4 might be required.

The closest approximation to an inert real soil that has been found to date is perhaps lunar soil.[1] This material, which has a gradation typical of terrestrial silty fine sands, has mechanical properties that are readily explainable and quantifiable in terms of void ratio and confining pressure. Differences in strength and compressibility among samples from different locations are explainable in terms of gradation, particle size, particle shape and particle composition (which controls size, shape and crushability). Since there is neither water nor significant gas in the lunar environment, there are no adjacent phases for the soil particles to interact with — hence, their inertness (except when in close contact with each other, which may result in small amounts of cohesion).

The actual relationships describing the mechanical properties of a terrestrial soil are rarely

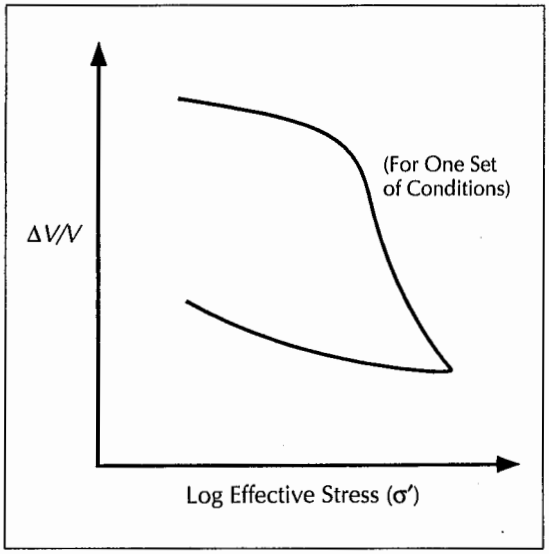


FIGURE 5. Idealized relationship for volume change (real soil).

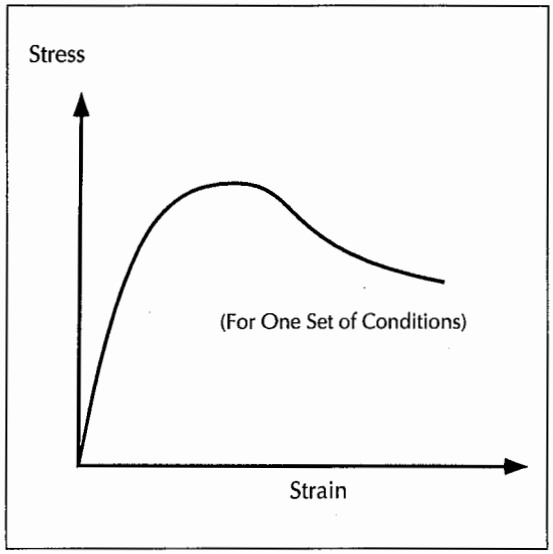


FIGURE 6. Idealized relationship for stress-strain (real soil).

as simple as those shown in Figures 1 through 4. More realistic representations for volume change, stress-strain, strength and hydraulic conductivity are shown in Figures 5 through 8. Parameters C, ϕ', E and A are seldom constants, although for engineering purposes they are often assumed to be so. Furthermore, the volume changes and deformations are usually time dependent, as a result of pore pressure adjustments and the viscous behavior of most soil structures. Fortunately, however, the variations are often at least understandable, if not quantitatively predictable, and the attempt to develop general constitutive models for behavior has occupied the attention of numerous researchers for many years.

Some Special Soil Behavior Phenomena. Not only does the mechanical behavior of real soils

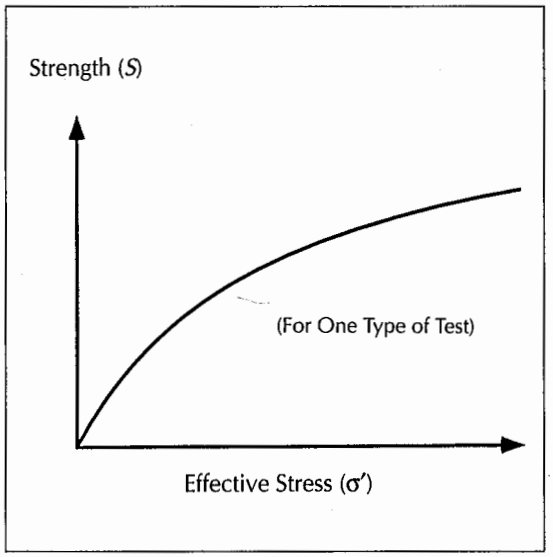


FIGURE 7. Idealized relationship for strength (real soil).

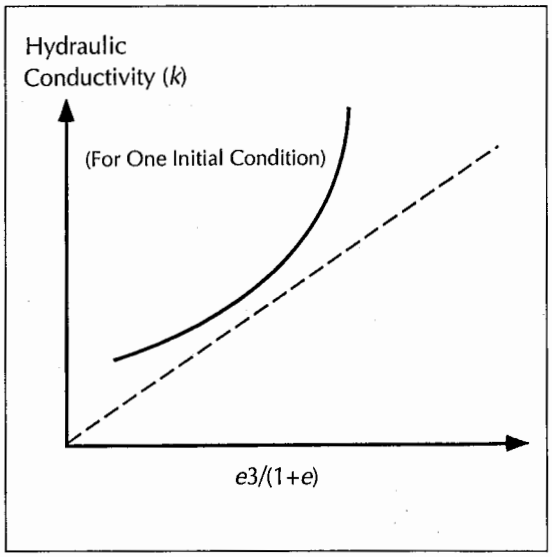


FIGURE 8. Idealized relationship for hydraulic conductivity (real soil).

deviate from that of ideal inert soils in the ways shown above, but there are numerous other phenomena that may be important, depending on the nature of a project or problem. Understanding them is the first step in developing suitable strategies for dealing with them. Some of these phenomena include:

Swelling Soils. Expansive soils are ubiquitous in many parts of the world, especially in semi-arid climates and where smectite group clay minerals are found. Figure 9 shows the swelling behavior of a clay soil compacted at the same water content and dry density by two different methods, thus producing two different initial structures. The magnitude of swell is significantly influenced by the structural difference. The figure shows also that the amount of swell decreases as the electrolyte concentration of the absorbed water increases. In addition, the data in Figure 10 indicate that the amount of swell is influenced by the stress path followed during unloading.

Collapsing Soils. Large areas of the earth's surface are covered by soils that are susceptible to large decreases in bulk volume when

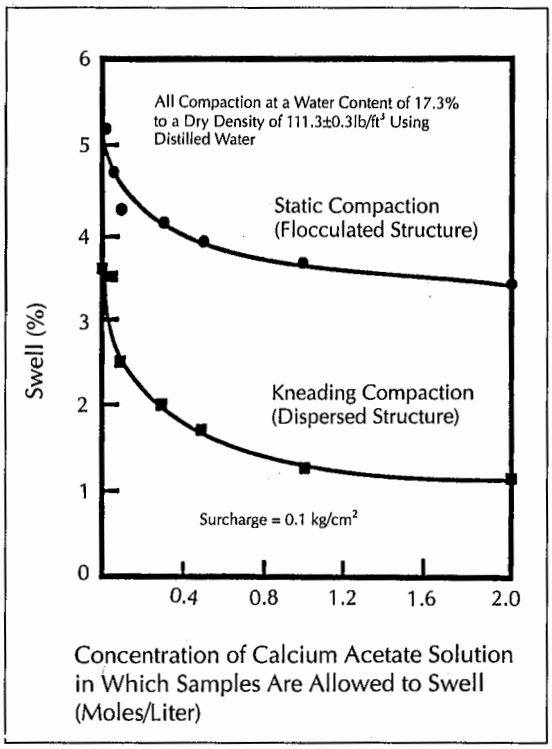


FIGURE 9. The effect of structure and electrolyte concentration of absorbed solution on the swell of sandy clay compacted by two methods.

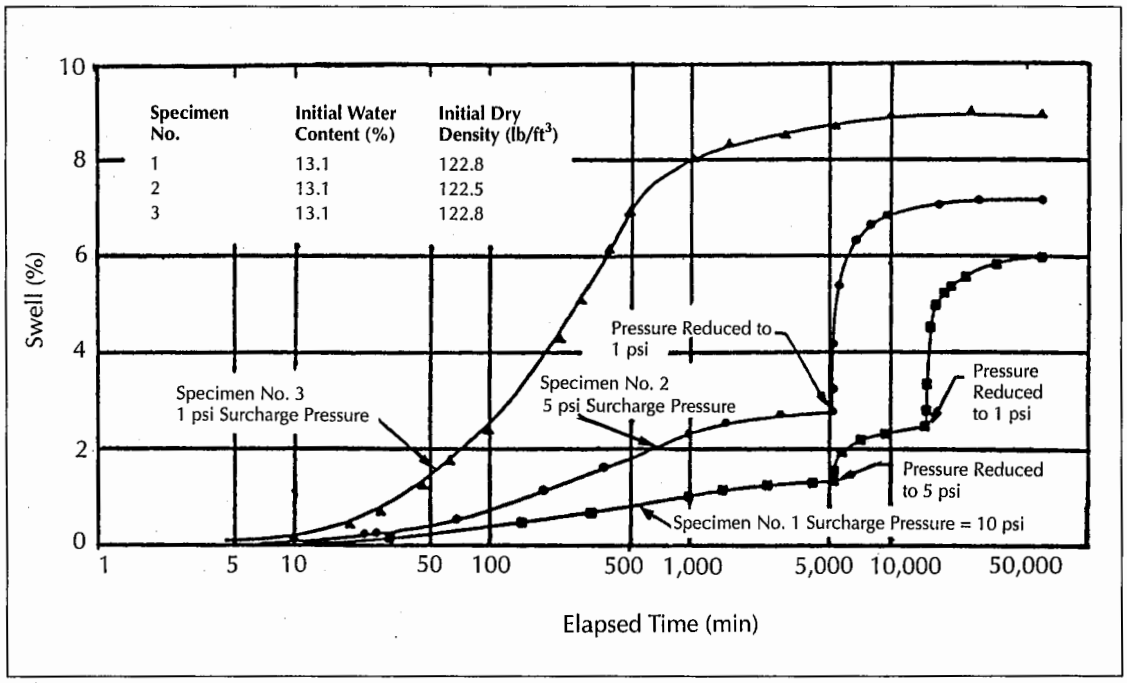


FIGURE 10. The effect of unloading stress path on the swelling of a compacted sandy clay.

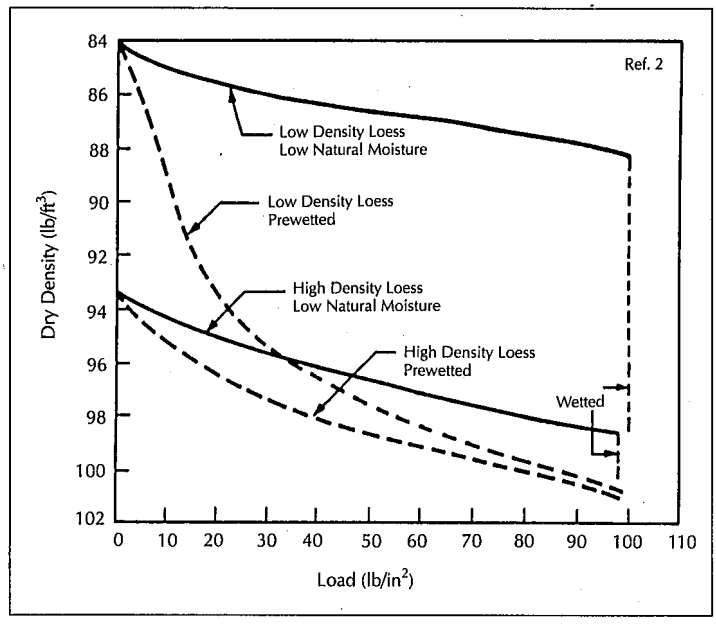


FIGURE 11. Collapse properties of Missouri River basin loess.

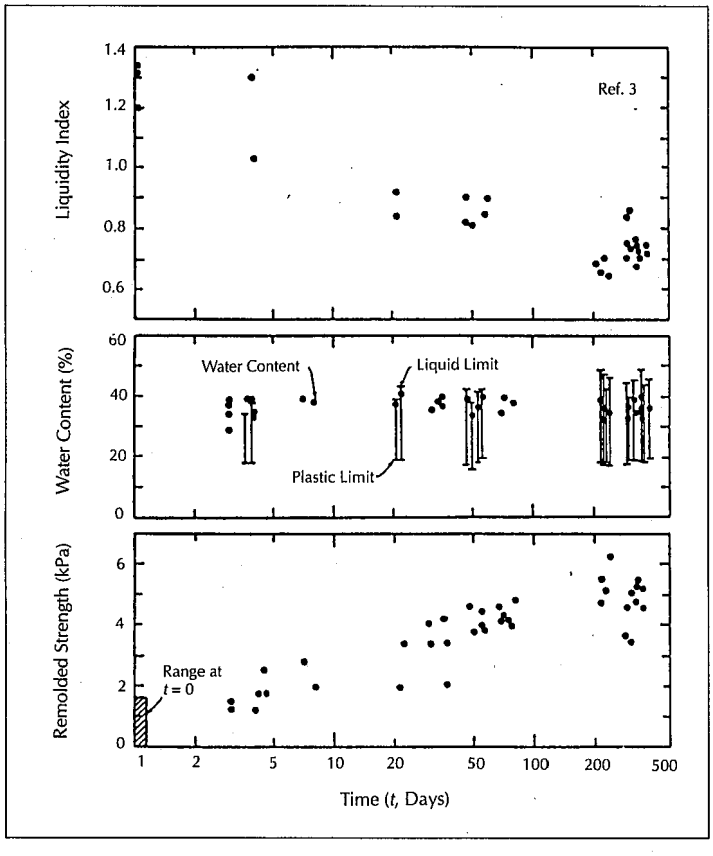


FIGURE 12. Changes in remolded strength and consistency of a Canadian quick clay as a function of time.

they become saturated. Collapse may be triggered by water alone or by saturation and loading acting together. Soils with collapsible grain structures may be residual, water deposited or aeolian. Settlement under increased loading may be small when the soil is at its natural moisture content, but after wetting it may be very large, as shown in Figure 11.

Dispersive Clays. Soils in which the clay particles will detach spontaneously from each other and from the soil structure and go into suspension in quiet water are termed dispersive clays. The consequences of the exposure of dispersive clays to water may include severe surface erosion and the formation of "badlands" topography and the formation of internal erosion tunnels in dams and dikes.

Slaking. Most intact fine-grained soils slake after exposure to air and subsequent immersion unconfined in water — *i.e.,* an initially intact piece of soil will disintegrate into a pile of pieces of sediment of small particles. This disintegration may begin immediately upon immersion or develop slowly with time. Slaking is usually more rapid and vigorous in materials that have been dried prior to immersion as compared to the same material immersed at its initial water content. The slaking of hard clays and clay shales is a concern in the stability of open excavations and the durability of shale is a concern when used as an aggregate or rockfill for construction.

Sensitive & Quick Clays. Sensitivity refers to the loss of

strength of an undisturbed clay when it is disturbed and remolded at constant water content. The strength loss may be so great that it will cause the remolded material to behave essentially as a fluid, in which case it is termed a *quick clay*. Early attempts to explain clay sensitivity led to the intensive study of physico-chemical phenomena in fine-grained soils by geotechnical engineers.

Aging of Quick Clays. Significant changes in the properties of quick clays can develop with time after sampling, including an increase in the remolded strength and liquid limit, and a decrease in the liquidity index without a change in water content. The changes in a remolded quick clay from Outardes-2 in Québec over a one-year period are illustrated in Figure 12. When changes such as these occur, the reliability of data obtained from tests on samples that have been stored for more than a few days after removal from the ground becomes questionable unless measures are taken to block the transformations responsible for them.

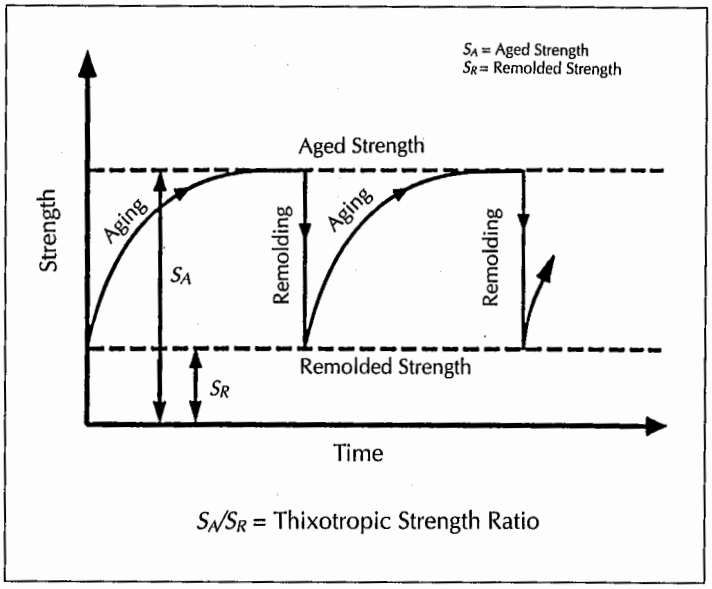


FIGURE 13. Properties of a purely thixotropic material.

Thixotropic Hardening. Thixotropy is an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby a material stiffens while at rest and softens or liquefies upon remolding. The properties of a purely thixotropic material are shown in Figure 13. Thixotropic hardening may account for low to medium sensitivity and for a part of the sensitivity of quick clays. It can be responsible for time-dependent increases in the strength and stiffness of compacted fine-grained soils. It may be important in influencing the flow properties of drilling muds and slurry wall materials.

Aging of Sands. Many sand deposits and sand fills exhibit "aging" effects wherein their strength and stiffness increase noticeably within periods of weeks to months after deposition, disturbance or densification. Illustrations of the effect of time on the penetration resistance of a hydraulic sand fill are shown in Figure 14, and of time on the penetration resistance of an in-situ sand after densification by blasting in Figure 15.

Creep & Stress Relaxation. Soils exhibit both creep and stress relaxation, as shown schematically in Figure 16. The magnitude of these effects increases with the plasticity, activity and water content of the soil. The form of behavior is essentially the same for all soils. Because of the viscous contribution to soil deformation, soil strength is influenced by rate of shear, as indicated by the data in Figure 17.

Coupled Flows. Fluids, electricity, heat and chemicals flow through soils. It has been well established that provided the flow process does not change the state of the soil, each flow rate, or flux, is related linearly to the driving force, or gradient, as shown in Figure 18. In addition, in most soils there are simultaneous flows of different types, even when only one type of driving force is acting. Thus, a driving force of one type can cause a flow of another type. A familiar example is electro-osmosis wherein a DC electrical gradient causes a hydraulic flow. Such flows are termed coupled flows. The different types of direct and coupled flows are listed in Table 1.

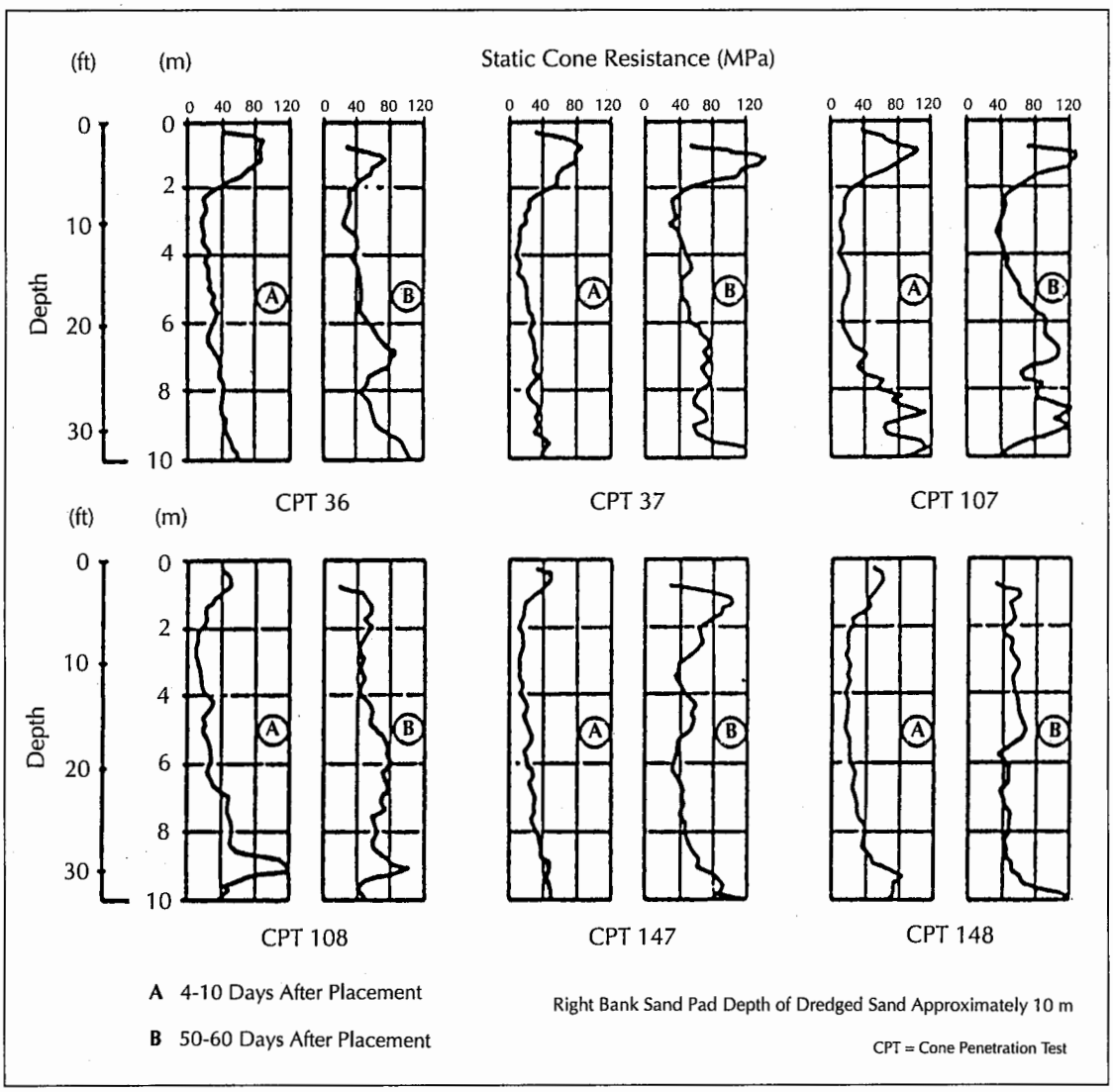


FIGURE 14. Penetration resistance profiles for a hydraulic fill at two times after placement.

Direct and coupled flows — especially advection, diffusion and electro-osmosis — are of particular current interest owing to their importance in geoenvironmental engineering problems.

Developing an Understanding of Soil Behavior

Phenomena of the type described in the preceding section cannot be explained in terms of an inert soil model. Rather, it is necessary to investigate in greater detail:

- Soil grain composition;
- The morphological characteristics of the particles; and,
- How the solid particles interact with the water and chemical environment around them, as well as with each other.

A detailed examination of these topics can be found in Mitchell.[5]

The engineering properties of any soil, and the special phenomena described above, depend on the composite effects of several interacting factors. These factors may be divided into two groups: *compositional factors* and *environmental factors*.

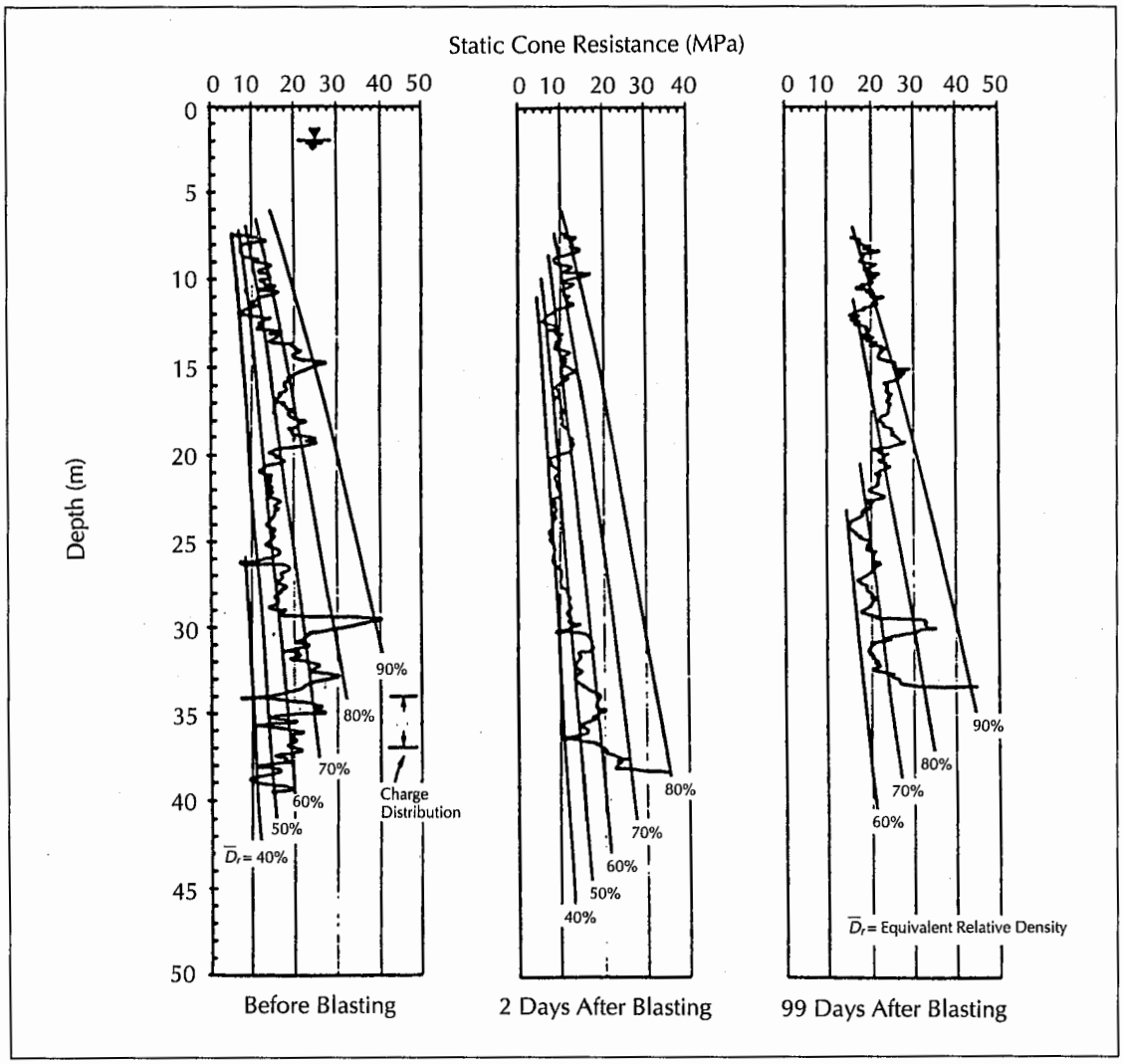


FIGURE 15. Cone penetration test records illustrating penetration resistance increase with time after blasting densification of clean sand at the Jebba Dam, Nigeria.

Compositional factors determine the potential range of values for any property and the extent to which any of the special phenomena could occur. Compositional factors can be studied using disturbed samples. Compositional factors include:

- Type of minerals;
- Amount of each mineral;
- Adsorbed cation types and amounts;
- Particle size, shape and size distribution;
- Pore water composition;
- Free and dissolved gases; and,
- Biological regime.

Environmental factors determine the actual value of any property. Undisturbed samples or in-situ measurements are required for the study of the influences of environmental factors on properties and behavior. Environmental factors include:

- Water content;
- Density;
- Confining pressure;
- Temperature;
- Fabric;
- Availability of water; and,
- Stress history.

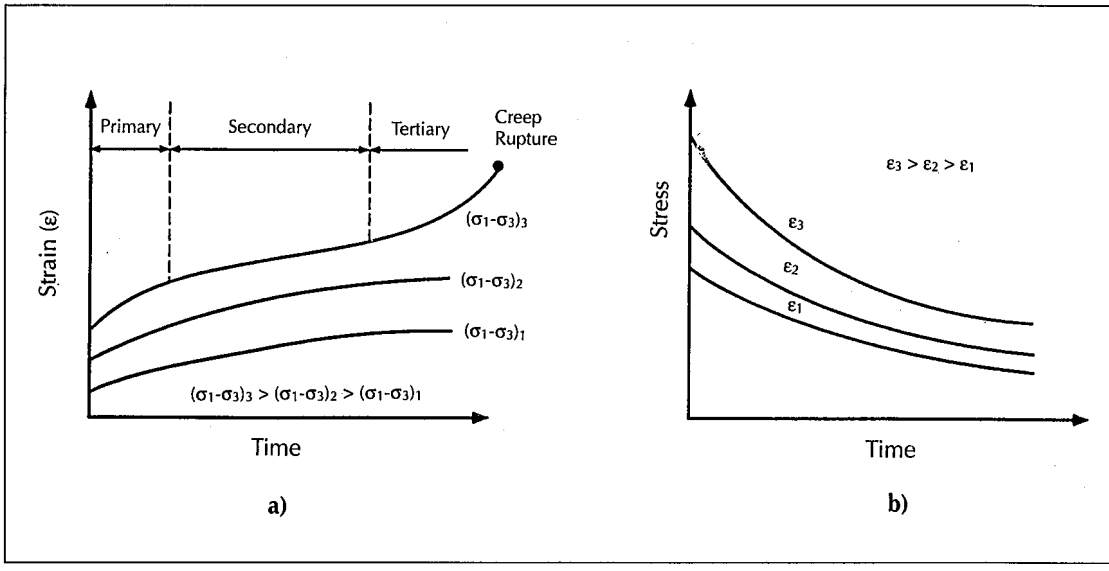


FIGURE 16. Creep and stress relaxation: a) creep under constant stress; and b) stress relaxation under constant strain.

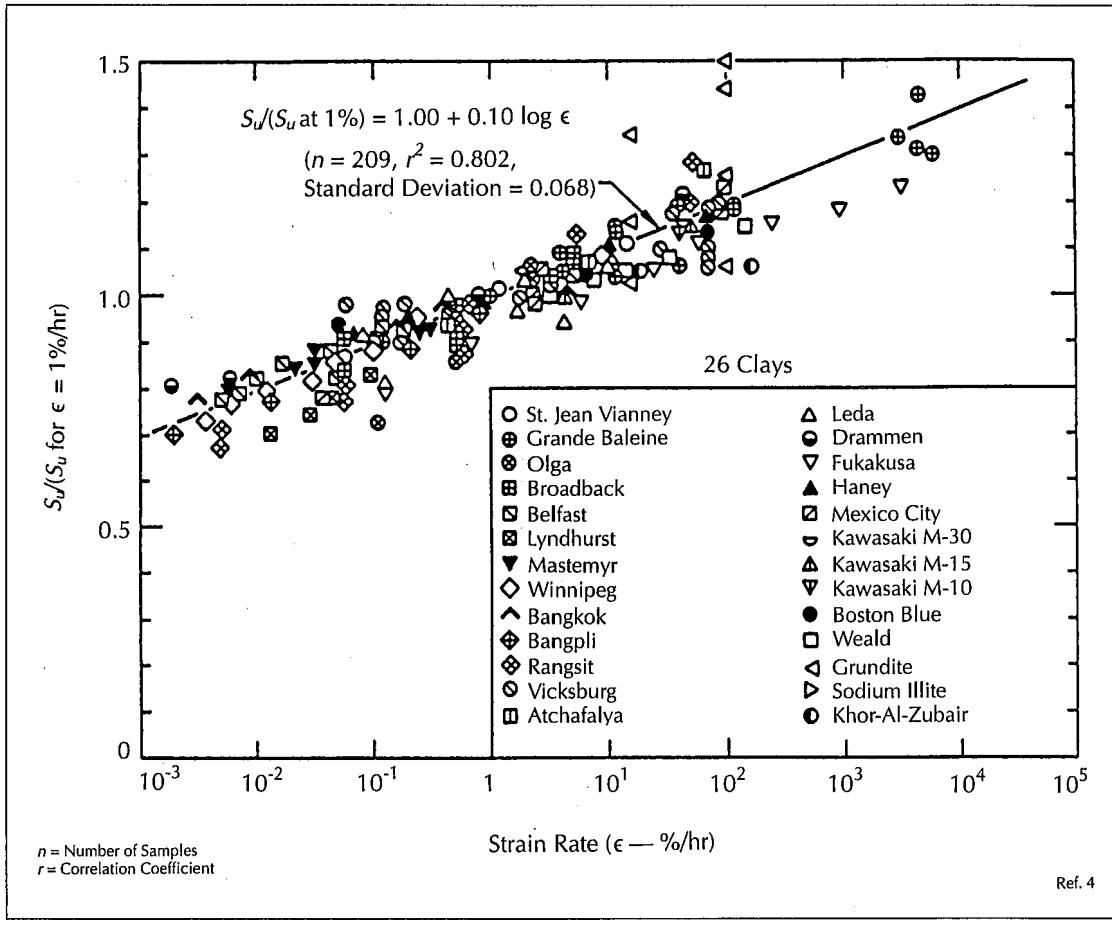
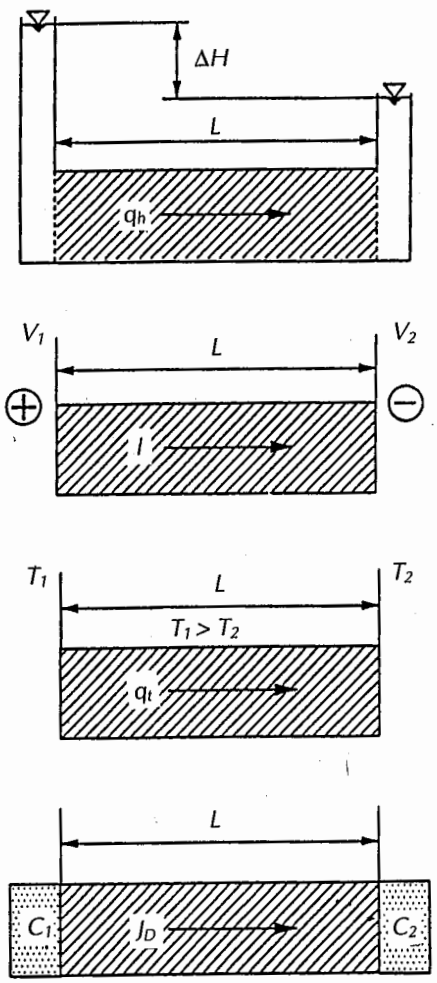


FIGURE 17. Effect of strain rate on undrained strength.

TABLE 1
Direct & Coupled Flow Phenomena

	Gradient (*X*)			
Flow (*J*)	**Hydraulic Head**	**Temperature**	**Electrical**	**Chemical Concentration**
Fluid	Hydraulic Conduction (Darcy's Law)	Thermo-osmosis	Electro-osmosis	Chemical Osmosis
Heat	Isothermal Heat Transfer	Thermal Conduction (Fourier's Law)	Peltier Effect	Dufour Effect
Current	Streaming Current	Thermo-electricity (Seebeck Effect)	Electric Conduction (Ohm's Law)	Diffusion & Membrane Potentials
Ion	Streaming Current	Thermal Diffusion of Electrolyte (Soret Effect)	Electrophoresis	Diffusion (Fick's Law)



Fluid

$q_h = k_h(\Delta H/L)A$

Darcy's Law

Electricity

$I = \sigma_e(\Delta V/L)A$

Ohm's Law

Heat

$q_t = k_t(\Delta T/L)A$

Fourier's Law

Chemicals

$J_D = D(\Delta c/L)A$

Fick's Law

In the relationships shown in this figure:

σ_e = Electrical Conductivity
q_s = Fluid Flow Rate
I = Electrical Current
q_t = Heat Flow Rate
J_D = Chemical Flow Rate by Diffusion
A = Cross-Sectional Area
L = Length
H = Head
V = Voltage
T = Temperature
c = Concentration
k_h = Hydraulic Conductivity
k_e = Thermal Conductivity
D = Diffusion Coefficient

FIGURE 18. Four types of direct flows in soils.

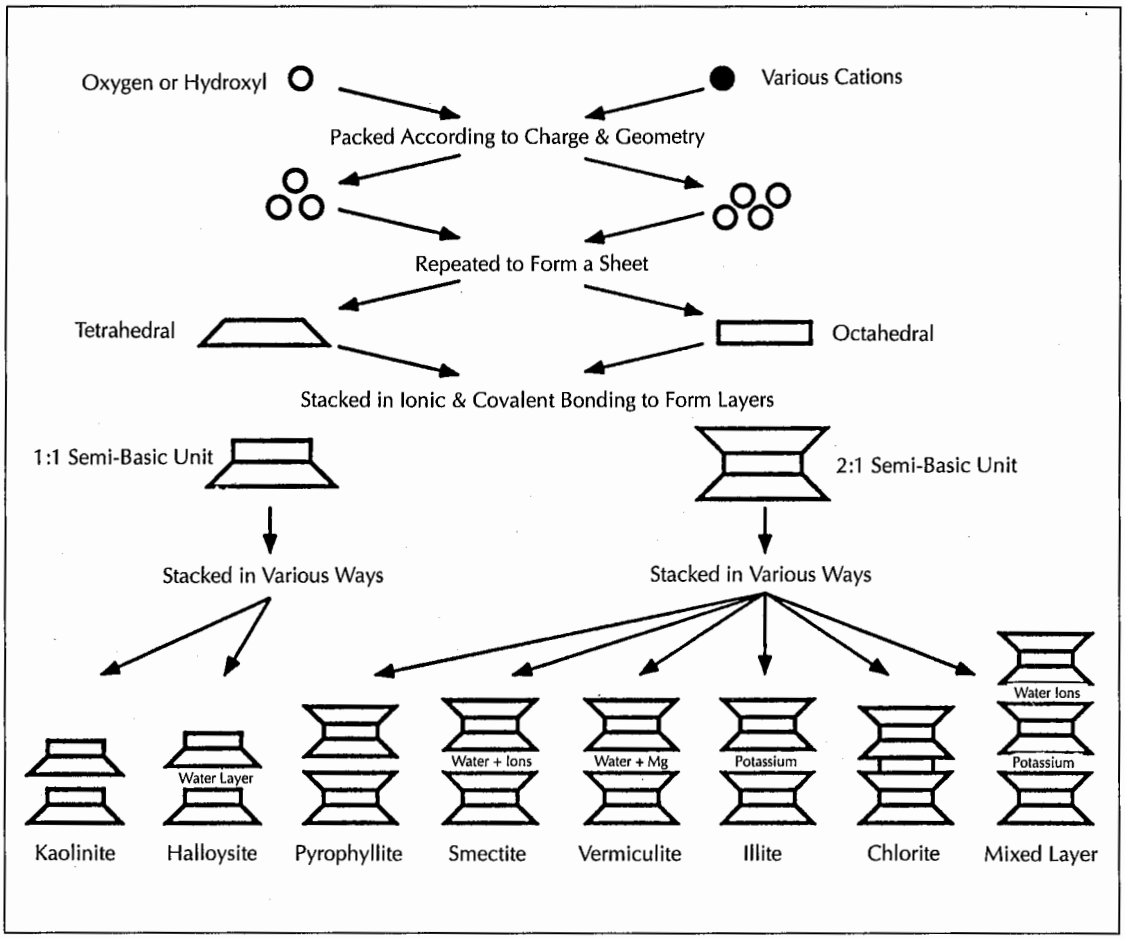


FIGURE 19. The basic structure of the clay mineral groups.

Soil mineralogy is fundamental in that it determines particle size, shape, surface area and the interface properties. Because the clay minerals are very small and they have a platy, tubular or lath-like shape, they have a very large specific surface area, ranging upwards to 800 m^2/gm. The general structures for the important clay minerals are shown schematically in Figure 19.

The surface layers of the clay minerals and most of the non-clay silicate minerals are composed of oxygen atoms in tetrahedral coordination held together by silicons. In the kaolin clays one of the surface layers of the platy particles is composed of hydroxyls. The surface oxygens and hydroxyls can be attracted simultaneously to silicons or other cations within the mineral particles or to hydrogens in the adjacent water phase. The resultant hydrogen bonding means that water is strongly attracted to soil mineral surfaces.

In addition, the mineral structures of the clay minerals are characterized by substantial isomorphous substitution, the end result of which is that the particles have a net negative electrical charge. As a result, cations are attracted to provide electrical neutrality. The quantitative expression of this electro-negativity and the amount of balancing cations is the cation exchange capacity (usually expressed in milliequivalents per hundred grams of dry clay). Important aspects of these conditions are that:

- The balancing cations can be of different types and valences;
- Cations of one type can be replaced by cations of another type; and,

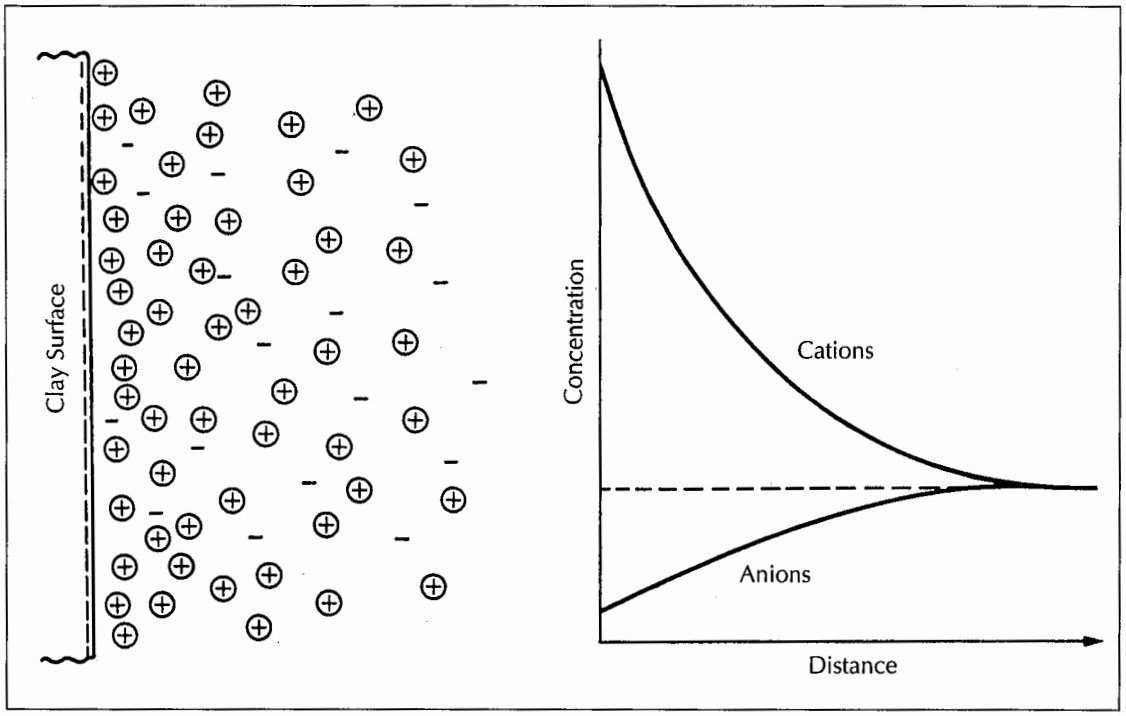


FIGURE 20. Distributions of ions adjacent to a clay surface according to the concept of the diffuse double layer.

- Cations of different types induce different properties.

Furthermore, in the presence of water, adsorbed cations dissociate from clay particle surfaces and form the diffuse part of a double layer of negative and positive charges that extends outward from particle surfaces, as shown in Figure 20.

Owing to their very small size and high specific surface area, clay particles behave as colloids, and many principles from colloid chemistry apply. The thickness and characteristics of this diffuse layer depend on the composition of the fluid phase as well as on the composition and structure of the mineral particles. Thick diffuse layers — which are associated with low cation valence, low electrolyte concentrations and pore fluids of high polarity and dielectric constant — produce strong repulsions between particles; whereas, high cation valence, high electrolyte concentration as well as low pore fluid polarity and dielectric constant are associated with small interparticle repulsions. The magnitude of the interparticle double layer repulsions relative to interparticle attractive forces (van der Waal's and electrostatic attractive forces) determines whether particles in suspension will flocculate or deflocculate. A wide range of particle associations is possible and the influences of different particle associations on the mechanical properties of a soil can be very large.

In summary, the small particle sizes, the high specific surface area, the attraction of water to soil particle surfaces and the net electrical negativity that leads to the formation of diffuse double layers sensitive to the surrounding environmental conditions mean that:

- Many different particle arrangements are possible for a soil at a given void ratio.
- Different particle arrangements at the same void ratio mean different effective stresses, different mechanical properties and different structural stability.
- Changed environmental conditions may or may not result in changes in the particle associations in a soil mass, depending on how tightly held the particles are. Subsequent mechanical disturbance of a

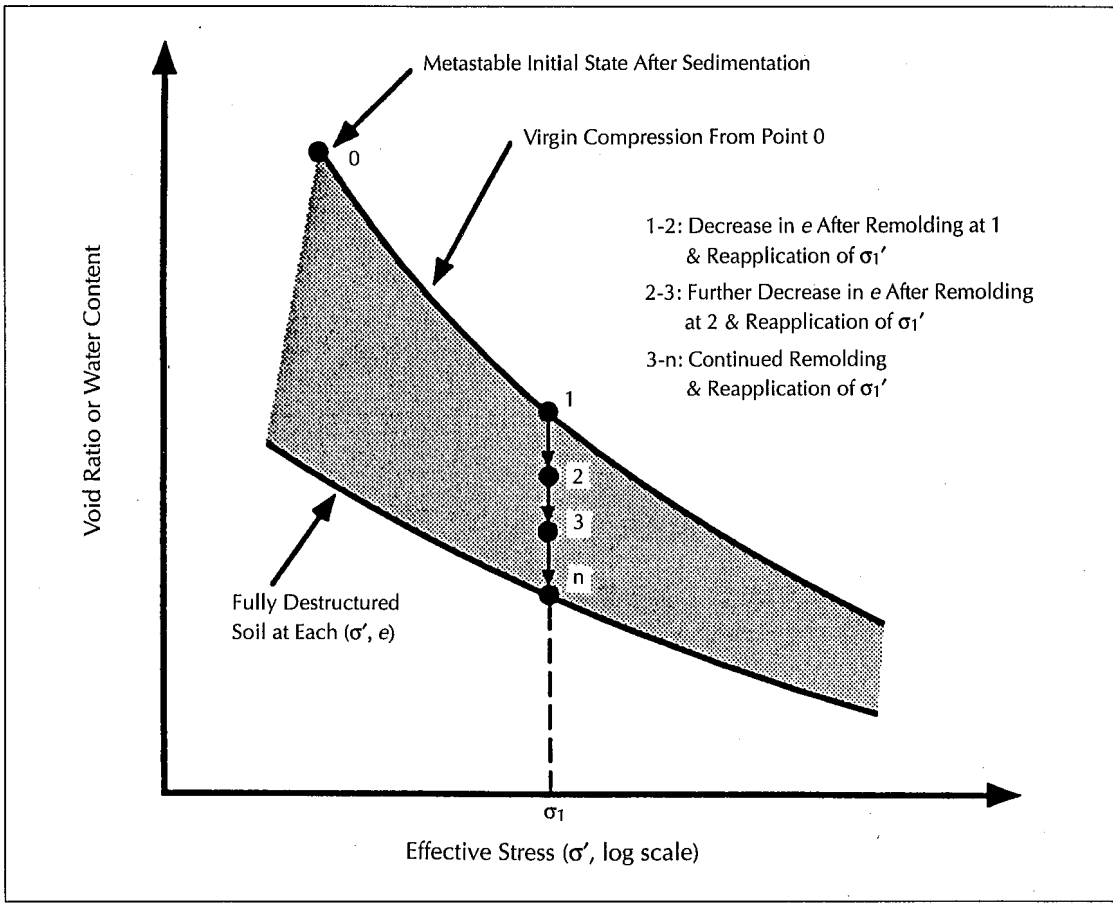


FIGURE 21. The influence of metastable fabric on void ratio under an effective consolidation pressure.

structure's environment that has been substantially changed since the initial structure formation may result in large changes in mechanical properties.

- The dynamic interactions between the solid and fluid phases of a soil mean that time-dependent reactions and interactions are possible.

Structure & Properties

The foregoing considerations can be combined with several principles that relate the fabric and structure of a soil to its mechanical properties in order to develop an understanding of real soil behavior and of the special soil behavior phenomena described earlier.

For any soil in which particles and particle groups flocculate, the initial fabric after soil stratum formation by weathering or sedimentation will be open and involve some degree of edge-to-edge and edge-to-face particle associations producing a fabric that can carry effective stresses at a void ratio higher than would be possible if the particles were arranged in the most efficient possible packing. If, under undrained conditions, the soil is mechanically remolded from a state such as represented by Point 1 in Figure 21, the fabric is disrupted, effective stresses are reduced because of the tendency for the volume to decrease, and the strength decreases.

If the original consolidation stresses are reapplied, then there will be additional consolidation and the void ratio will decrease to a point as represented by Point 2 in Figure 21. Subsequent mechanical remolding and reapplication of stresses will cause consolidation to Point 3, and continued repetition of the process

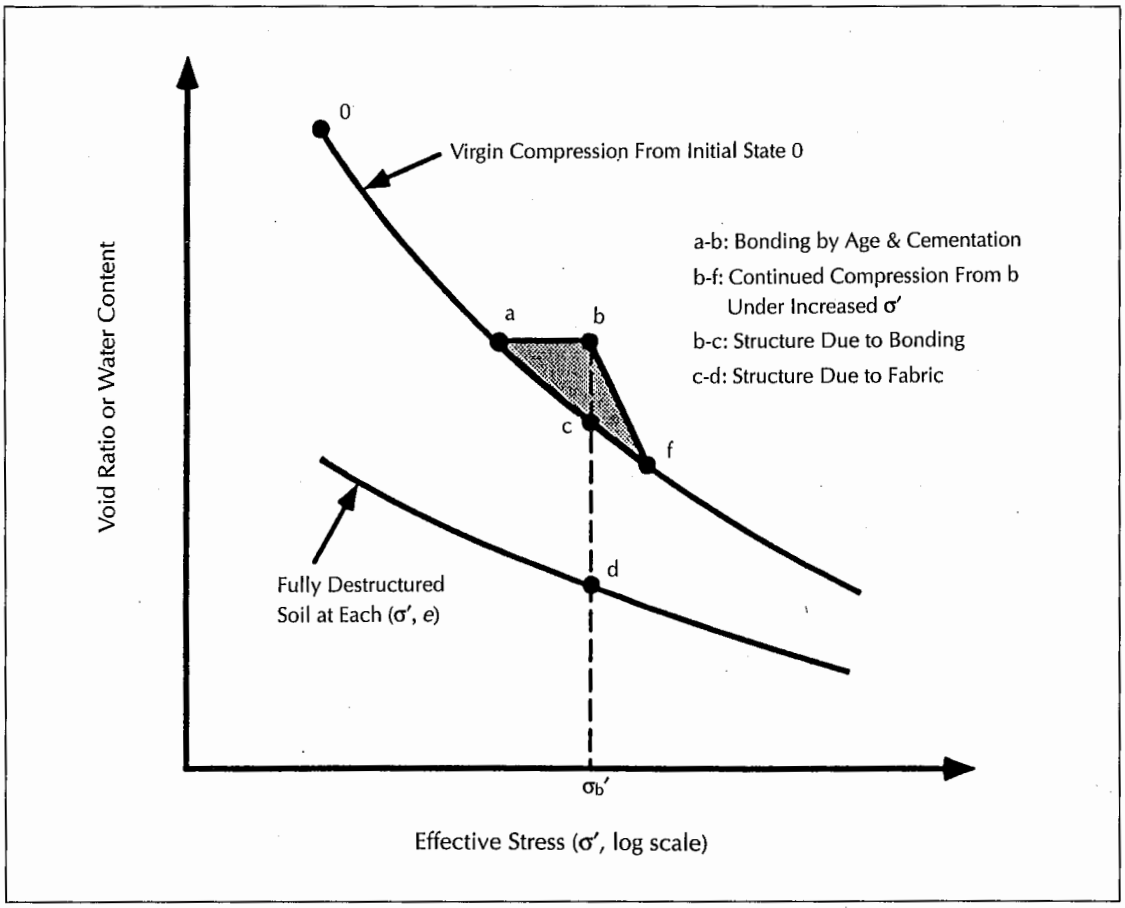


FIGURE 22. Possible states in void ratio-effective stress space.

will lead ultimately to a minimum void ratio for the fully destructured soil at *n*. Thus, if the soil is at any state within the shaded area depicted in Figure 21, it will have some degree of metastability of structure.

Also, it is possible that a soil can be at a state to the right of the virgin compression curve in Figure 21 as a result of bonding by chemical cementation or aging effects. Thus, the full range of possible states in void ratio-effective stress space is greater (see Figure 22) than shown in Figure 21. Virgin compression from an initial state at *0* to *a* is followed by the development of bonding, which enables the soil to resist additional compressive stress, *a* to *b*. At Point *b*, the soil is under effective stress, σ_b'. The completely destructured soil under the same stress would be at Point *d*. The difference in void ratios between the structures soil at *b* and the destructured soil at *d* is made up of a bonding contribution, *b* to *c*, and a fabric contribution, *c* to *d*.

Several principles relate the fabric and structure of a soil to its mechanical properties. Some of these principles can be used, along with an understanding of the colloidal and chemical behavior of the clay-water-electrolyte system, to explain the special soil behavior phenomena presented earlier:

- Under a given effective consolidation pressure, a soil with a flocculated fabric is less dense than the same soil with a deflocculated fabric.
- At the same void ratios, a flocculated soil with randomly oriented particles and particle groups is more rigid than a deflocculated (destructured) soil.
- Once the maximum precompression stress has been reached, a further incre-

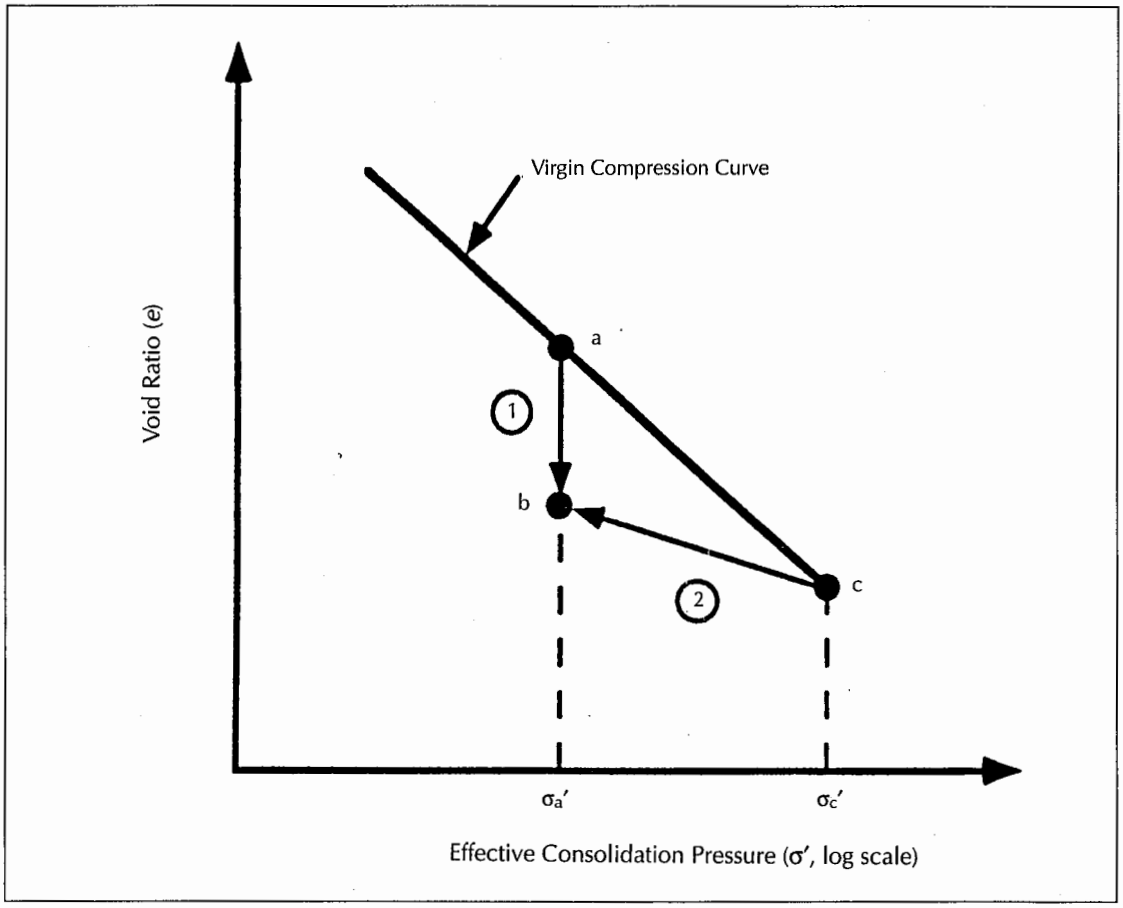


FIGURE 23. Different paths to reach the same void ratio-effective stress state.

ment of pressure causes a greater change in the fabric of a flocculated soil structure than in a deflocculated soil structure.

- At a given porosity, the average pore diameter and range of pore sizes is smaller in deflocculated and/or destructured soils than in flocculated and/or undisturbed soils.
- Shear displacements usually align platy particles and particle groups with their long axes in the direction of shear.
- Anisotropic consolidation stresses tend to align platy particles and particle groups with their long axes in the major principal plane.
- Stresses are not usually distributed equally among all particles and particle groups. Some particles and particle groups may be essentially stress free as a result of arching by surrounding particle groups.
- For an uncemented soil, two samples can have different structures at the same void ratio-effective stress coordinates if they have different stress histories. In Figure 23, a sample initially at Point *a* on the virgin compression curve can deform to Point *b* as a result of disturbance and reconsolidation or by secondary compression (creep) under stress σ_a' sustained for a long time. A sample initially at *c* can reach Point *b* as a result of unloading from σ_c'. The stress-deformation properties of the two samples will differ.
- Volume change tendencies determine pore pressure development during undrained deformation.
- Changes in the structure of a saturated soil at constant volume are accompanied by changes in effective stress. These effective stress changes are immediate.

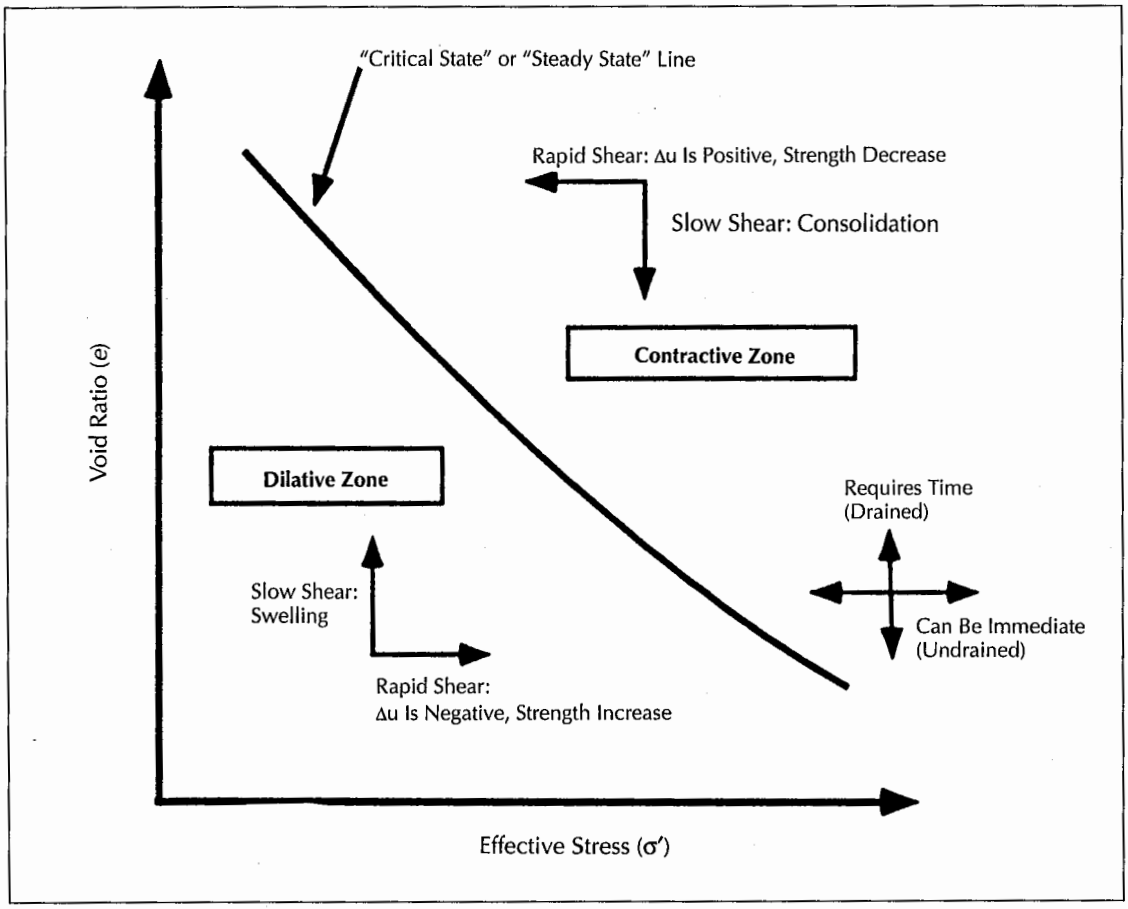


FIGURE 24. Initial state in relation to the critical or steady state line and its influence on pore pressures and volume change during deformation.

- Changes in the structure of a saturated soil at constant effective stress are accompanied by changes in void ratio. The change in void ratio is not immediate, but depends on the time for water to drain from or enter the soil.

The last three principles are illustrated by Figure 24. For any saturated, destructured soil the unique relationship between combinations of void ratio and effective consolidation pressure is commonly referred to as the *critical state* or *steady state* line. If the soil is on this line, there is no tendency for a change in volume during shear deformation. However, if the state of the soil is in the region above and to the right of this line, it will either contract if the rate of deformation is slow, or positive pore pressure will be generated if deformation is rapid. On the other hand, if the soil is initially at a state in the dilative zone, slow deformation will be accompanied by swelling, and rapid deformation will be accompanied by the generation of negative pore pressures. In general, normally to slightly overconsolidated clays and saturated loose sands are contractive; whereas, heavily overconsolidated clays and dense sands are dilative.

Understanding Unusual Soil Behavior

Nine special types of soil behavior, ranging from swelling and collapsing soils through quick clays and thixotropic hardening to coupled flows, were described above. Some fundamental considerations were reviewed in the previous section that can provide an explanation and understanding for these special types of soil behavior.

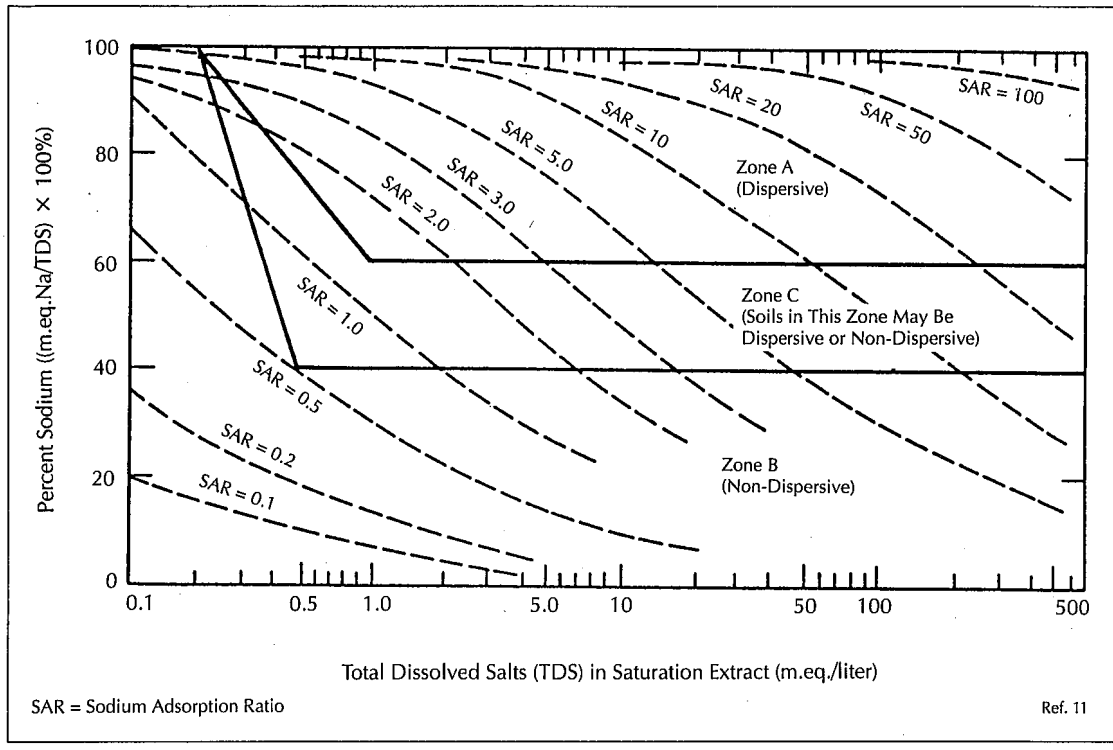


FIGURE 25. The relationship between dispersibility and dissolved pore water salts based on pinhole tests and field observations.

Swelling Soils. Theories for clay swelling have been developed along two rather different lines. The first, based on interparticle repulsions arising from double layer interactions, was introduced into geotechnical engineering literature by Bolt,[6] and has been extensively evaluated since, as summarized by Mitchell.[5] This theory is often referred to as the *osmotic pressure theory of swelling*. The second considers that swelling pressures and swelling are caused mainly by surface hydration, and it has been developed mainly by Low.[7,8]

Purely mechanical contributions that result from the rebound of deformed particles and particle groups when confining pressures are reduced should be added to those possible causes of swelling. As shown by Terzaghi, simple mixtures of sand and mica flakes can be made to duplicate the compression and rebound behavior of a range of different naturally occurring fine-grained soils.[9] That mechanical factors are significant in swelling is illustrated also by the differences in total swell measured for samples of clay compacted by different methods to different initial structures as shown in Figure 9.

Without getting into the details and limitations of each approach, an explanation that is consistent with both the influences of the double layer/osmotic pressure theory and the water adsorption theory can be outlined as follows.

Clay particle charge density, adsorbed cation type, pore fluid electrolyte concentration and pore fluid dielectric constant determine the proportions of fully expandable and partially expandable layers in a potentially swelling clay. For example, calcium montmorillonite does not swell to interplate distances greater than about 0.9 nm (at which the particles stabilize). This particle structure is stabilized by attractive interactions between the basal planes of the unit layers. In contrast, the individual unit cell layers in sodium montmorillonite (bentonite) may separate completely in low electrolyte concentration solutions. In the presence of high electrolyte concentrations or pore fluids of low dielectric constant, interlayer

swelling is suppressed. In such cases, the effective specific surface is greatly reduced relative to that for the case where interlayer swelling occurs. As a result, the amount of water required to satisfy surface hydration is reduced greatly.

For example, data from Low suggest that a hydration water layer thickness on smectite surfaces of about 10 nm is needed to reach a distance beyond which the water properties are no longer influenced by surface forces,[7] and the swelling pressure of montmorillonite is about 100 kPa for a water layer thickness of about 5 nm.[10] For a fully expanding smectite having a specific surface of 800 m^2/gm, this latter water layer thickness would correspond to a water content of 400 percent. Thus, a material such as sodium montmorillonite would be expected to be expansive over a very wide range of water contents, and experience shows clearly that it is.

On the other hand, consider an illite or a smectite made up of partially expanded clay particles (termed *quasi crystals*) so that interlayer swelling is negligible. Since both clays have surface structures that are essentially the same, it would be expected that the hydration forces should be similar. Thus, an adsorbed water layer thickness of 5 nm would also be reasonable. However, the specific surface areas of pure illite and nonexpanded smectite are only about 100 m^2/gm, which corresponds to a water content of 50 percent for a 5-nm thick water layer. For a pure kaolinite having a specific surface of 15 m^2/gm, the water content would be only 7.5 percent for a 5-nm thick adsorbed layer.

It is evident, therefore, that the specific surface dominates the amount of water required to satisfy forces of hydration. Except for very heavily overconsolidated clays and those soils that contain large amounts of expandable smectite, there is sufficient water present even at low water contents to satisfy surface hydration forces, and swelling is small. On the other hand, when the clay content is high and particle dissociation into unit layers is extensive, the effective specific surface area is large and swelling can be significant. The tendency for smectite clays to dissociate into unit layers can be evaluated through consideration of double layer interactions, with those conditions that favor the development of high repulsive forces leading to greater dissociation. The interlayer bonding is too strong in the other clay mineral groups (see Figure 19) for separation of particles into individual unit layers so that the surface area would be large enough to give high swell or swell pressure.

Collapsing Soils. Collapsing soil behavior such as that exhibited by loess, debris flow material and other soils with a loose and metastable structure (as illustrated by the test results in Figure 11) can be understood on the basis of the considerations in the previous sections (see especially Figures 22 and 24). Owing to deposition in a loose state — *i.e.*, the contractive zone depicted in Figure 24 — and the subsequent development of weak bonding by clay or light cementation at silt and sand grain contacts, the soil reaches condition *b* in Figure 22. Then, exposure to water can cause a decrease in void ratio along *b* to *c* under self weight, or along *b* to *f* if additional loading is applied.

Dispersive Clays. The susceptibility of a fine-grained soil to dispersive clay behavior is not indicated by usual soil classification tests such as grain size distribution and Atterberg limits. The stability of clay soil structure against breakdown and particle dispersion, at least for nonmarine clays, is better indicated by the proportion of sodium in the adsorbed cation complex. Chemical and physical tests for the identification of dispersive clays have been developed that are based on such determinations as the sodium concentration in relation to the total dissolved salts in the pore water (see Figure 25)[11] and the pinhole test. In the pinhole test distilled water is allowed to flow through a 1.0-mm diameter hole drilled through a compacted specimen. The water becomes muddy and the hole rapidly increases in size in dispersive clays. Additional tests include the Soil Conservation Service Dispersion Test in which the percentage of particles finer than 5 μm is determined by hydrometer analyses of samples with and without the use of a dispersing agent. The higher the ratio of percentage material finer than 5 μm by weight measured in the test without a dispersing agent to that measured in the test with a dispersing agent, the greater the probability of dispersion will be in the field.

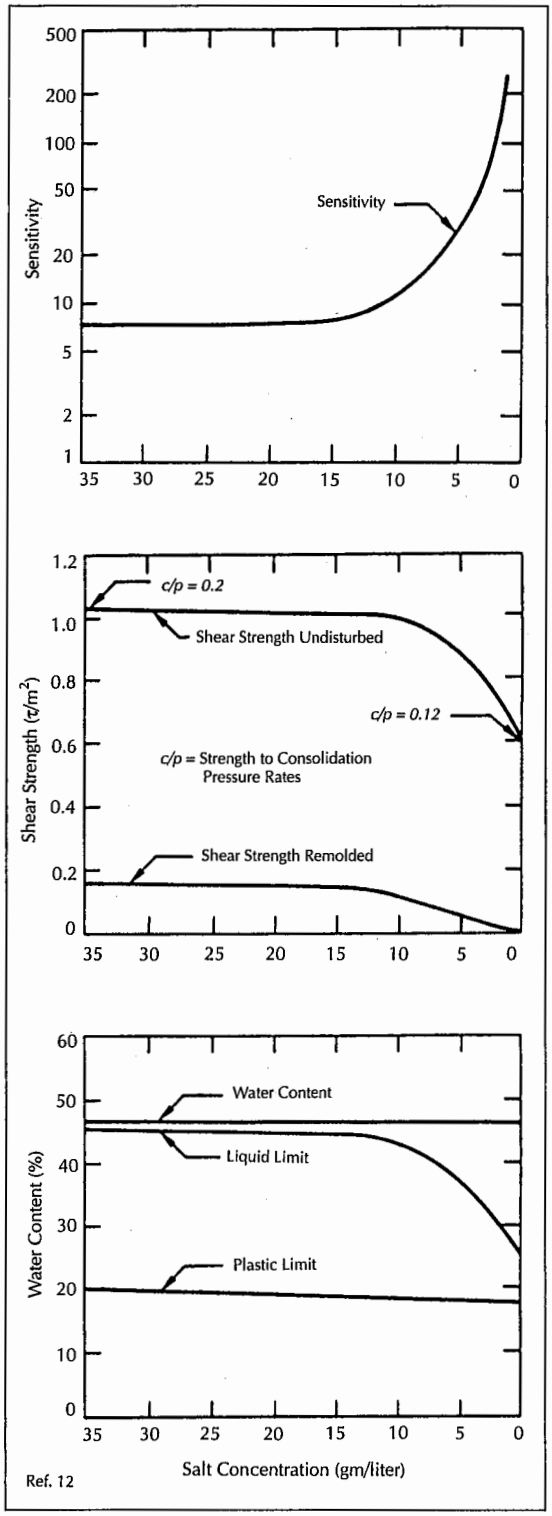


FIGURE 26. Changes in properties of a normally consolidated marine clay as a result of freshwater leaching.

Unfortunately, however, the tests as designed do not always reliably classify soils as to their dispersivity. In reality, this is not surprising, because whether or not a soil will exhibit dispersive behavior depends not only on its chemical and mineralogical composition, but also on its state (as reflected by water content, density and structure), on the chemistry of the water to which it is exposed and on the specific conditions of exposure such as temperature, confining pressure and velocity of flowing water. It is important, therefore, that in testing for dispersivity the sample state and water chemistry duplicate to the greatest extent possible those conditions to be expected in the field.

Slaking. There are three mechanisms responsible for slaking. Dispersion, which depends on the clay and water chemistry, was described above. Swelling, resulting from stress relief and water adsorption due to surface hydration and osmotic forces, can weaken the structure and lead to disintegration. Compression of entrapped air in partly saturated materials as a result of rapid water absorption can exert tensile stresses on the structure leading to splitting.

Sensitive & Quick Clay Formation. Most very sensitive and quick clays are post-glacial marine clays. Reduction in salinity, usually by leaching, is the first step in their development. Leaching by freshwater occurs after a drop in sea level or rise in land level. The presence of percolating water in sand and silt lenses is sufficient to remove salt from the clay by diffusion without requiring the water to flow through the pores of the intact clay. Although this leaching causes little change in fabric, the interparticle forces may be changed, resulting in a change in undisturbed strength of up to 50 percent, and such a large reduction in the remolded strength that a quick clay may form. The large increase in interparticle repulsion results from the decrease in electrolyte concentration and is responsible for deflocculation and dispersion of the clay on mechanical remolding. The changes in properties that accompany reductions in salt concentration in a normally consolidated marine clay are shown in Figure 26.

The details of the formation of a quick clay are somewhat more complex than indicated by

a simple reduction in salt content.[5,13] However, the necessary conditions for the formation of a quick clay may be stated: they are low salt content, high percentage of monovalent cations in the adsorbed layers on the clay particles, and high pH.

Quick Clay Aging. Changes in the remolded strength and consistency of a Canadian quick clay as a function of time were shown in Figure 12. These aging effects can be explained as follows.

When a quick clay is sampled or exposed, some contact with air and oxygen is inevitable. Small amounts of organic matter present in the clay are oxidized to form carbonic acid which, in turn, dissolves calcium carbonate, thus increasing the concentrations of calcium and bicarbonate in the pore water. Oxidation of pyrite (small amounts are present in the rock flour that comprises much of the solid content of quick clays) forms sulfuric acid and ferric hydroxide. The sulfuric acid reacts with magnesium calcite to increase the concentrations of dissolved calcium and magnesium in the pore water and in the adsorbed cation complex on the clay particles. Sodium and potassium are displaced from the double layer to the pore water.

The chemical changes developed in LaBaie, Québec, quick clay as a function of time are illustrated in Figure 27.[13] The different curves in Figure 27 refer to different conditions of sample storage. The salinity increase and the increase in concentrations of the divalent cations cause increases in the remolded strength and the liquid limit as well as decreases in the sensitivity and liquidity index.

The aging of quick clays is an excellent example of how even seemingly small changes in environmental conditions can result in significant changes in properties. These changes can occur over times typical of those associated with the field and laboratory phases of many projects — *i.e.*, several weeks to a few months. To minimize aging effects, the exposure of samples to air should be minimized, thick wax caps should be used with rust-free sample tubes, and samples should be stored at low temperatures to slow down reaction rates.

Thixotropic Hardening. A basic outline of thixotropic hardening (see Figure 13) first should be presented. Sedimentation, remolding, and compaction of soils produce structures compatible with the processes that are acting on the soils. Once the externally applied energy of remolding or compaction is removed, however, the structure is no longer in equilibrium with the surroundings. If the interparticle force balance is such that attraction is in excess of repulsion, there will be a tendency for the flocculation of particles and particle groups and for a reorganization of the water-cation structure to a lower energy state. Both effects, which have been demonstrated experimentally, take time because of the viscous resistance to particle and ion movement.

Several studies have shown that there is a continual decrease in pore water pressure, or an increase in pore water tension, with time after compaction or remolding. The concurrent increase in effective stress accounts for the observed increase in strength.

Aging of Sands. The effect of time on the penetration resistance of a hydraulic fill was depicted in Figure 14, and the effect of time on the penetration resistance of a natural sand deposit after densification by blasting was shown in Figure 15. Unfortunately, the specific mechanisms responsible for strength and stiffness increases associated with the aging of sands are not yet known in detail.

There has not been clear resolution of the relative importance of chemical factors (such as silicate precipitation at interparticle contacts and changes in surface characteristics) and mechanical factors (such as time-dependent stress redistribution and particle reorientations) in causing the observed behavior. Evidence and arguments in support of chemical hardening mechanisms are given by Mitchell and Solymar;[14] whereas, Mesri *et al.* and Schmertmann present a case for mechanical mechanisms, particularly secondary compression-like processes.[15,16] More recent analyses and experiments have failed to yield unambiguous results concerning mechanisms,[17] and reliable means for predicting the magnitude and rate of strength and stiffness increase remain to be developed.

Creep & Stress Relaxation. The general characteristics of creep and stress relaxation were shown in Figure 16, and the effect of shear rate

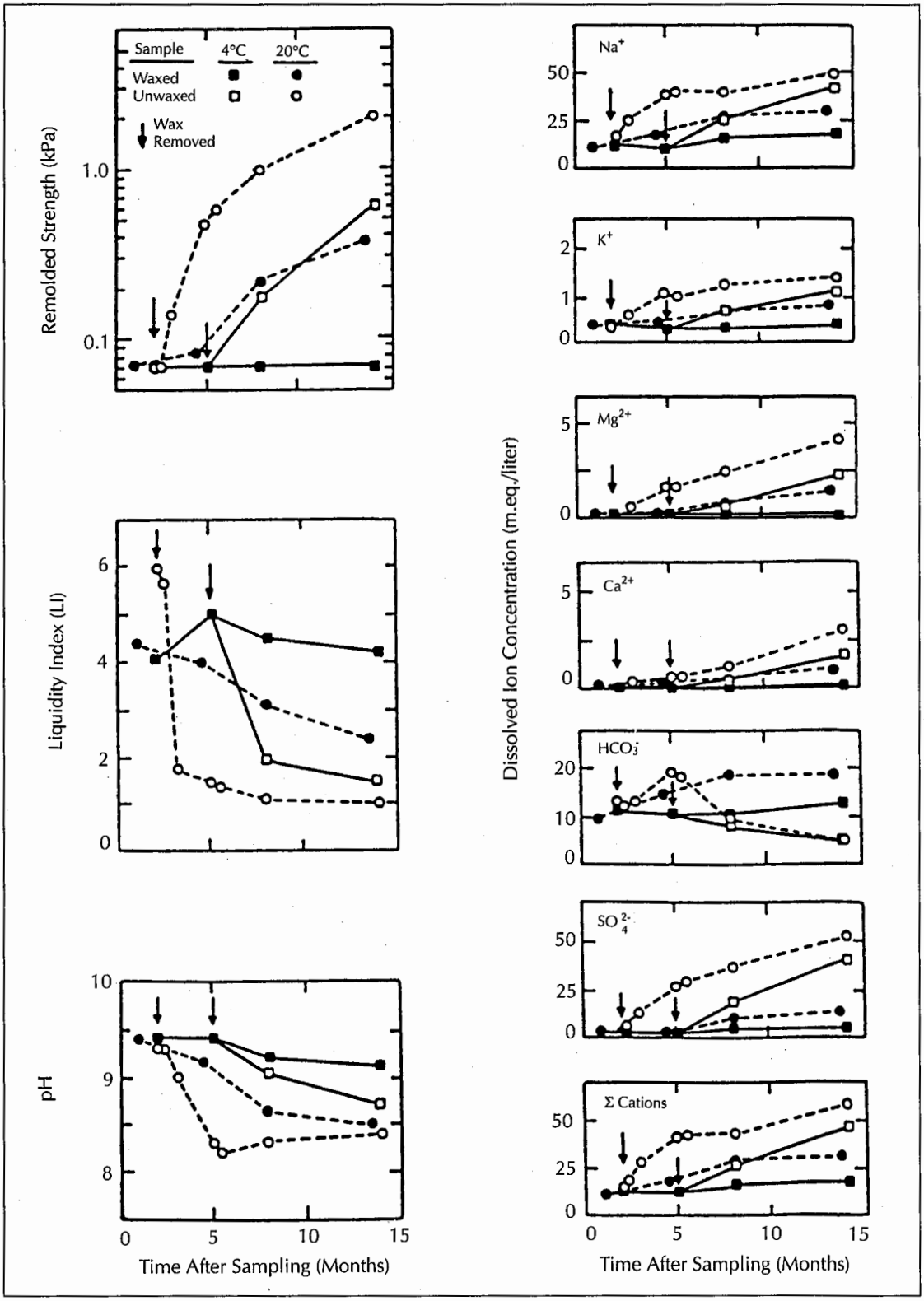


FIGURE 27. The effect of time and storage conditions on properties of LaBaie quick clay.

on shear resistance was portrayed in Figure 17. The results of many studies have revealed that these time-dependent changes in stress and strain are the result of thermally activated processes. Analyses within the framework of rate process theory have provided insights into the stress and time dependencies of creep and stress relaxation rates. Such studies have also enabled development of consistent hypotheses for bonding, effective stresses and strength of soils that are suitable for most soil types, soil states and loading conditions. A detailed review of this subject may be found in Mitchell.[5]

Coupled Flows. The last of the special soil behavior phenomena described earlier was coupled flow processes. Types of coupled flow processes that may occur in soils were summarized in Table 1. A comprehensive treatment of coupled flows in terms of mechanisms, theories and applications can be found in Mitchell.[5,18]

Coupled flows are generated because of the non-uniform distributions of cations and anions and electrical charges (as shown in Figure 20) and because of the small pore sizes in fine-grained soils. Clays can exhibit membrane properties, which means that the passage of certain ions and molecules through the clay may be restricted in part or in full at both microscopic and macroscopic levels. These restrictions mean that the application of gradients of different types — hydraulic, thermal, electrical or chemical — can induce local and global imbalances in pressure, temperature, electrical potential and/or chemical concentration. These imbalances result in flows of different types in addition to the direct flow of the type corresponding to the applied gradient.

Some recent applications of coupled flow considerations in geotechnical problems are described in Mitchell[18] and include thermally driven moisture flow and its consequences relative to the thermal stability of buried electrical cable backfills, in-situ determination of clay consolidation properties by electro-osmosis, the relative importance of diffusive and advective chemical transport through clays, waste containment and site remediation by electro-osmosis, potential consequences of in-situ potentials in relation to slope stability, and the possibility for stabilizing the foundations of existing structures using electro-osmosis.

Conclusion

In most geotechnical engineering projects and problems, correct site characterization and property evaluation are the two most critical elements. If they are not done reasonably and reliably, neither an understanding of or confidence from subsequent soil mechanics analyses can be obtained, no matter how sophisticated the analyses may be or how powerful the computer that performs them. Thus, running through every aspect of our work is the need for sufficient understanding of what soils are and the principles that govern their important geotechnical properties, which in most cases are those pertaining to flows through them, to volume change, to stress-deformation behavior and to strength.

Real soils are not composed of inert materials, but their state and properties depend importantly on their composition and the environmental conditions in which they exist. Simple soil models should be used for analysis only with an appreciation of the assumptions and approximations that they contain. Some unusual, but generally well-known types of soil behavior have been described herein that are important in practice, but which, while understood, cannot be represented by simple equations or empirical correlations. Nonetheless, it is important that their consequences be considered in practice.

No one has stated the importance of soil mechanics in geotechnical engineering better than Professor Arthur Casagrande. In his closing remarks to an engineering conference held by the Corps of Engineers in 1938, Casagrande said:

> *Soil mechanics is a complex subject because the materials with which we deal are so utterly complex. In addition, to penetrate deeper and deeper into the mysterious behavior of soils, we need a much greater variety of tools than, for example, are required in structural engineering. A man who desires to work in soil mechanics needs not only a thorough knowledge of the properties of materials in general, of mathematics, and at least the principles of the theory of elasticity,*

but also knowledge of physical chemistry and geology, and in addition the mastery of some entirely new conceptions and avenues of approach which form the fundamentals of soil mechanics.

NOTE — *This article was adapted from the annual Arthur Casagrande Lecture delivered by the author to BSCES on October 21, 1993.*

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Chemically Enhanced Wastewater Treatment: An Alternative & Complement to Biological Wastewater Treatment

The addition of metal salts and polymers in the wastewater treatment process can produce treatment benefits without incurring a significant increase in capital cost.

INGEMAR KARLSSON &
SHAWN P. MORRISSEY

Two types of chemically enhanced wastewater treatment (CEWT) processes — chemically enhanced primary treatment (CEPT) and direct precipitation — which add metal salts and polymers at the beginning of the treatment process provide the means to supplant or augment biological wastewater treatment. The primary difference between these CEWT processes is the amount of chemicals added to the system.

CEPT has been used since the early 1980s in southern California and Canada (see Figure 1 and Table 1). It is usually accomplished by requiring minimal additional construction to conventional primary treatment plants. Therefore, little capital cost is required to convert a primary treatment plant to a chemically enhanced primary treatment plant.

The treatment consists of adding metal salts (such as ferric chloride or aluminum sulfate) and/or cationic polymers and an anionic polymer to the waste stream to enhance settling. The metal salts, which are used as a coagulating agent, need rapid mixing in order to optimize the coagulation process. At concentrations of 20 to 30 mg/l, they should be added as far upstream of the sedimentation tanks as possible

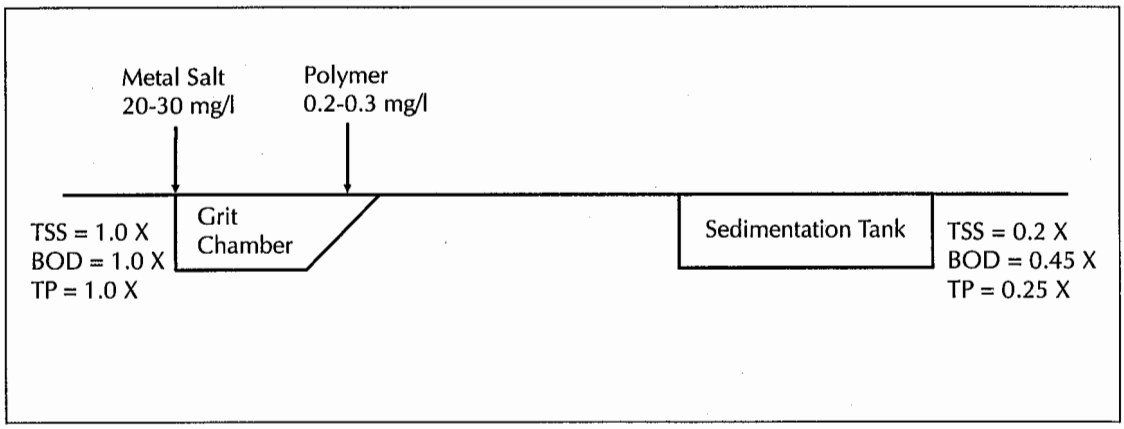

FIGURE 1. Schematic of the CEPT process.

to allow enough mixing time for the coagulation process to occur. The anionic polymer (0.2 to 0.3 mg/l) needs rapid mixing initially to dilute the polymer, then gentle mixing to promote flocculation and the formation of large settleable flocs. It should be added ahead of the sedimentation tanks. CEPT has typically achieved 80 percent total suspended solids (TSS), 50 to 60 percent biochemcial oxygen demand (BOD) and 75 percent total phosphorus (TP) removal.[1]

Direct precipitation is being used in Norway and Sweden in plants designed to optimize the chemical treatment process for phosphorus removal (see Figure 2 and Table 2). The treatment consists of approximately 150 mg/l of a metal salt. The difference between it and CEPT is that in the direct precipitation process, phosphorus removal is a primary objective; hence, the use of the large concentration of metal salts. Direct precipitation consistently achieves 90 percent TSS, 85 percent BOD and 90 percent phosphorus removal.[2]

The Effect of CEWT on Subsequent Treatment Processes

Carbon Removal. For very sensitive receiving waters, the quantity of organic matter in the effluent from wastewater treatment plants has to be very low. This requires that the chemical

TABLE 1
Summary of Treatment Efficiency & Chemical Addition for Various Advanced Primary Treatment Plants

Location	Performance					Chemical Addition		
	Flow (mgd)	BOD (mg/l) Influent	BOD (mg/l) Effluent	TSS (mg/l) Influent	TSS (mg/l) Effluent	Type	Concentration (ppm)	Duration
Pt. Loma San Diego	191	276	119	305	60	$FeCl_3$ Anionic Polymer	35 0.26	Continuous
Orange County Plant 1	60	263	162	229	81	$FeCl_3$ Anionic Polymer	20 0.25	8 hrs (Peak Flow)
Orange County Plant 2	184	248	134	232	71	$FeCl_3$ Anionic Polymer	30 0.14	12 hrs (Peak Flow)
JWPCP Los Angeles County	380	365	210	475	105	Anionic Polymer	0.15	Continuous
Hyperion Los Angeles	370	300	145	270	45	$FeCl_3$ Anionic Polymer	20 0.25	Continuous
Sarnia Ontario, Canada	10	98	49	124	25	$FeCl_3$ Anionic Polymer	17 0.3	Continuous

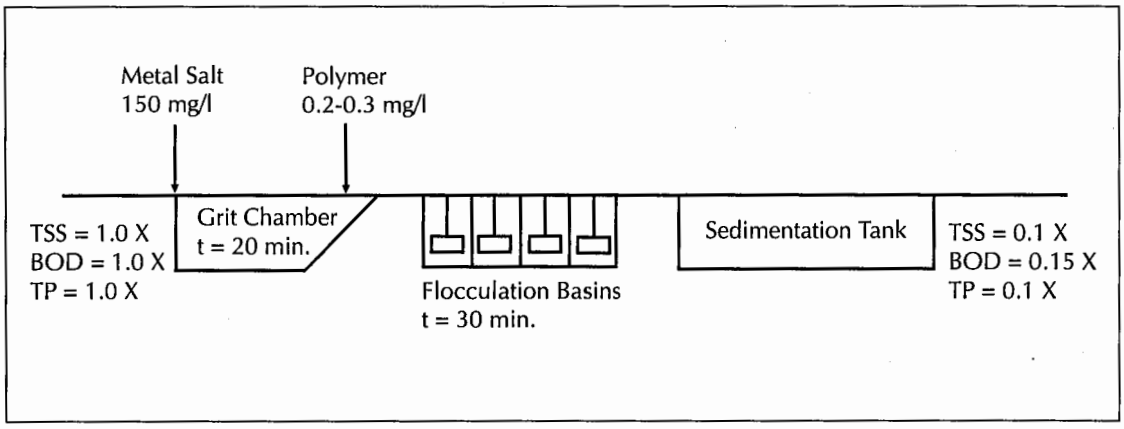

FIGURE 2. Schematic of the direct precipitation wastewater treatment process.

precipitation step be supplemented with a biological step.

In some cases, with direct precipitation alone, it is possible to reach BOD reductions comparable to those obtained from a conventional biological treatment plant at lower costs and by using less space. In addition, the phosphorous reduction will be 70 percent higher. Direct precipitation can also be used for unloading an overloaded conventional biological plant.

Consider the economic aspects of CEPT. Figure 3 shows how organic material is distributed in a conventional biological treatment plant. Of an incoming amount of 75 grams of BOD per person per day (g BOD/pe-d), 30 percent is separated in the primary clarifier, 60 percent in the biostage and the remaining 10 percent goes into the receiving water. The 60 percent converts into 43 grams biodegraded in the biostage. To biodegrade 1 kilogram of BOD requires about 1.3 kWh. (A conventional activated sludge biological treatment plant utilizes large amounts of electrical energy to produce and disperse into the aeration basin the oxygen necessary to reduce the BOD.) Power consumption is then about 20 kWh per person per year for a conventional biological treatment plant.

With CEPT, 60 percent of the organic substance is separated in the primary clarifiers and 30 percent, or 23 grams of BOD, goes to the biological stage (see Figure 4). Only 11 kWh per person per year is required — a 45 percent reduction in the energy demand of a conventional system.

Nitrification. Scandinavia and the United States now have new regulations limiting the discharge of nitrogen into receiving waters. The force of these regulations creates a problem for wastewater treatment plants because most have insufficient biological volumes and reten-

TABLE 2
Summary of Treatment Efficiency & Chemical Addition for Direct Precipitation Plants in Norway

Location	Performance							Chemical Addition		
	Flow (mgd)	P_{tot} (mg/l) Influent	P_{tot} (mg/l) Effluent	BOD (mg/l) Influent	BOD (mg/l) Effluent	TSS (mg/l) Influent	TSS (mg/l) Effluent	Type	Concentration (ppm)	T_{mix} (min)
Average of 23 Plants From Norway		5.5	0.5	216	42	172	27	Lime/$FeCl_3$ Anionic Polymer	100-200 0-0.2	20 30
Oslo	62	2.9	0.14	140	30	120	10	$FeCl_3$ Anionic Polymer	150 0.2	15 30

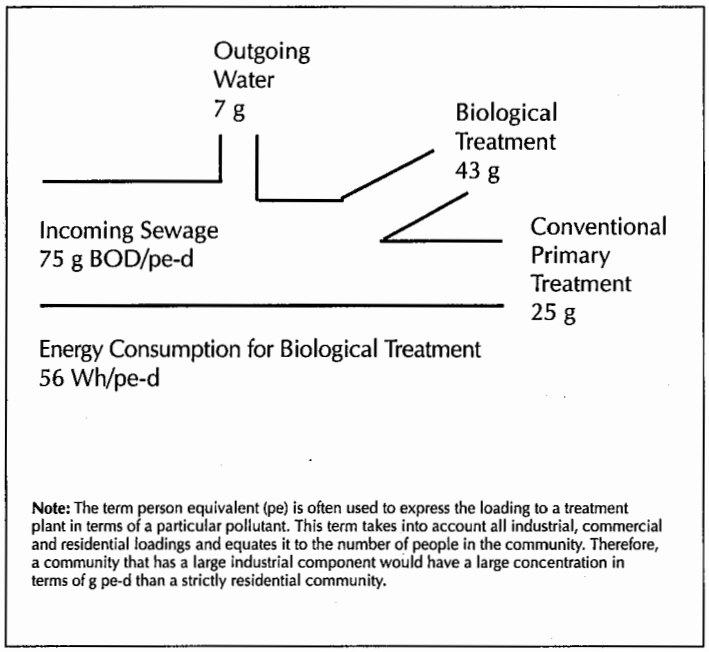

FIGURE 3. The separation of organic matter in conventional biological treatment.

tion times to achieve the required nitrification and denitrification. CEWT techniques can be applied to this problem with very favorable results.

To remove nitrogen biologically, two separate stages are necessary — an aerobic and an anoxic stage. An anoxic stage differs from an anaerobic stage in that oxygen, bound as a nitrate, is present. In the aerobic stage, ammonia in the wastewater is oxidized to nitrite and then to nitrate in a process referred to as nitrification, while in the anoxic stage the nitrate is reduced to a gaseous form in a process referred to as denitrification.

With a trickling filter nitrifiers start to grow when most of the organic matter is consumed and the number of fast-growing heterotrophic bacteria (organisms that use organic carbon for the formation of cell tissue) start to decrease. Large volumes and surface areas are necessary if the BOD/N ratio is high. The lower the BOD/N ratio, the smaller the surface area in the trickling filter required to achieve nitrification.[3]

For an activated sludge process, sludge age is one of the most important parameters for nitrification. In order to build up a nitrifying system, it is necessary to increase the sludge age enough so that slow-growing autotrophic bacteria (organisms that derive cell carbon from carbon dioxide) are not washed out of the biological system. During winter, with low water temperatures (often below 10°C), it is necessary to further increase the sludge age.

An increased removal of organic matter (BOD) and suspended solids (up to 100 percent) over the primary system can be expected when CEPT is used. This situation will in fact double the sludge age, if the same aeration tank volume and sludge concentration is used as illustrated by the following design equation for completely mixed activated sludge with recycle:

$$SA = BOD/(F/M * X)$$

where:

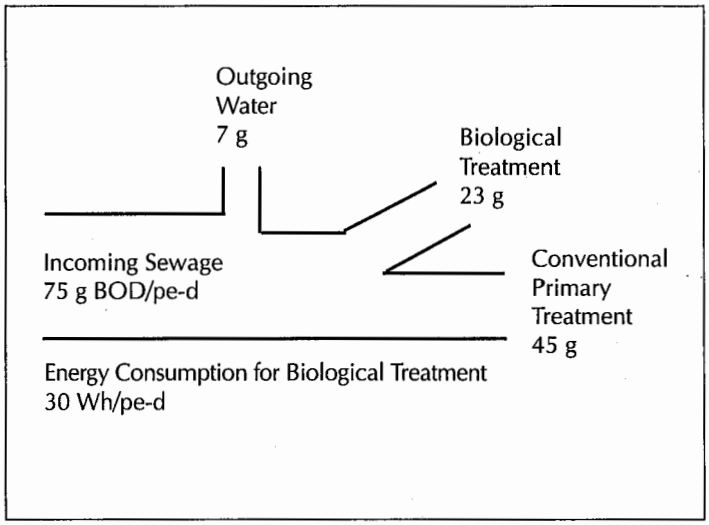

FIGURE 4. The separation of organic matter using CEPT followed by biological treatment.

SA = Sludge age (days)
BOD = influent BOD concentration (mg/l)
F/M = Food-to-microorganism ratio (1/day)
X = Solids concentration in aeration tank (mg/l)

In addition, the organic matter is mainly in a dissolved, readily denitrifiable form that will further increase the sludge age based on changes in kinetic parameters. The use of CEPT thus enables nitrification without requiring aeration tank expansion.

Denitrification. In the second stage of nitrogen removal (denitrification) the nitrate produced in the first stage (nitrification) is reduced to a gaseous form of nitrogen and is released to the atmosphere. Generally, the gaseous form is nitrogen gas (N_2) but it can also be nitrous or nitric oxide. This transformation can be accomplished by a wide variety of microorganisms, which under anoxic conditions use nitrate as their electron acceptor in place of oxygen. These bacteria are classified as facultative heterotrophic bacteria, due to their ability to utilize either oxygen or nitrate in their oxidation of organic matter.

Nitrogen removal treatment processes can be classified as post-denitrification and pre-denitrification. These processes are differentiated by the point at which denitrification occurs. In post-denitrification, the wastewater is first nitrified aerobically, then passed to the anoxic zone. Because the denitrifying bacteria require a carbon source for growth, water that has already received primary, and some secondary, treatment may be too depleted in carbon, in which case a carbon source such as methanol or hydrolyzed sludge must be added to the anoxic zone.

In pre-denitrification, the wastewater flows first through the anoxic zone, where denitrifiers can take advantage of the highest possible carbon concentration. The water then passes to an aerobic zone, and nitrified water produced at that point is then recirculated to the anoxic zone for denitrification.

One important factor influencing the pre-denitrification process is the BOD/N ratio. Opposite to the nitrification process, a high ratio is favorable. CEPT decreases the BOD/N ratio; however, the remaining fraction of organic substances shows the highest oxidation rate since they are highly soluble. This fraction corresponds to the soluble content of BOD and will have the same level as water without chemical treatment.

TABLE 3
Typical Wastewater Composition

TSS	200 mg/l	80 g pe-d
BOD	200 mg/l	80 g pe-d
SBOD*	50 mg/l	20 g pe-d
COD**	450 mg/l	180 g pe-d
TP	7 mg/l	2.8 g pe-d
N	35 mg/l	14 g pe-d

Notes: * SBOD = Soluble BOD
** COD = Chemical Oxygen Demand

If the readily denitrifiable organic matter in the wastewater is too low to fulfill the nitrogen removal demands, it is possible to use the precipitated CEPT sludge that has a volume less than one percent of the wastewater being treated (containing about 75 percent of the organic matter in the wastewater). Thus, there is a good possibility that this sludge can be treated in a separate optimized process and that it can be hydrolyzed into an easily degradable form. The denitrification process can be accelerated and additional volume can be saved. The use of CEPT to accelerate the nitrification and for hydrolysis of the sludge to accelerate the denitrification process has been developed and implemented commercially.[4] The hydrolysis process can be performed in different ways, the most interesting being biological anaerobic fermentation and thermal hydrolysis.

Sludge & Energy Production for Carbon Removal

A typical wastewater has the composition as shown in Table 3. Sludge production has been calculated for different wastewater treatment processes. The suspended solids, including some of the BOD plus the contribution from the coagulant, removed from the primary settling tanks constitutes the primary sludge. In the

TABLE 4
Comparison of Various Treatment Processes

	CEPT	DP*	CPT**+AS***	CEPT+AS	DP+AS
BOD Removed	60%	75%	90%	90%	90%
TSS Removed	80%	90%	90%	90%	90%
TP Removed	75%	90%	20%	75%	90%
Retention Time	2 hrs	4 hrs	12 hrs	7 hrs	9 hrs
Energy Produced	120 Wh/pe-d	150 Wh/pe-d	180 Wh/pe-d	180 Wh/pe-d	180 Wh/pe-d
Energy Consumed	0	0	51 Wh/pe-d	35 Wh/pe-d	29 Wh/pe-d
Net Energy	120 Wh/pe-d	150 Wh/pe-d	129 Wh/pe-d	145 Wh/pe-d	151 Wh/pe-d
Digested Sludge	46 g/pe-d	73 g/pe-d	51 g/pe-d	55 g/pe-d	77 g/pe-d

Notes: * DP = Direct Precipitation
** CPT = Conventional Primary Treatment
*** AS = Activated Sludge Secondary Treatment

biological step, the conversion of organic matter, mostly soluble BOD, into removable form constitutes the secondary sludge. Sludge digestion produces gas equivalent to 2.5 Wh per gram of BOD.

For all examples, the "typical" wastewater characteristics are as shown in Table 3 with references made to person equivalence. Sludge production is calculated assuming a 50 percent reduction in BOD during digestion.

Table 4 summarizes parameters of the various conventional and CEWT treatment processes — removal percentages, retention times, energy produced and consumed, and disgested sludge quantities. Figures 5 through 9 illustrate these key parameters. In

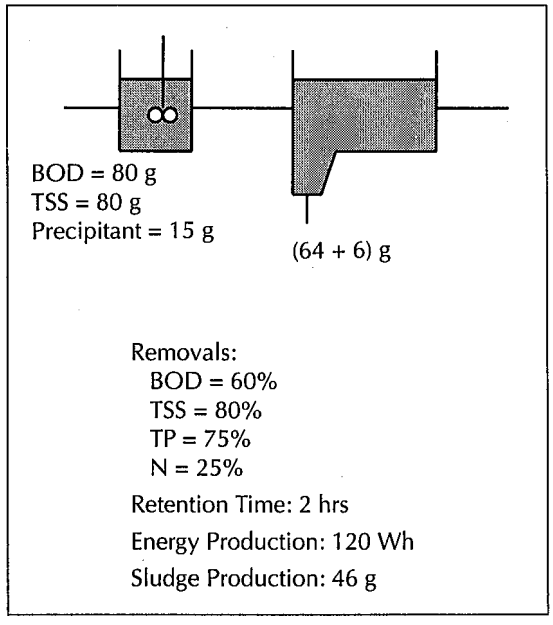

FIGURE 5. CEPT sludge and energy production.

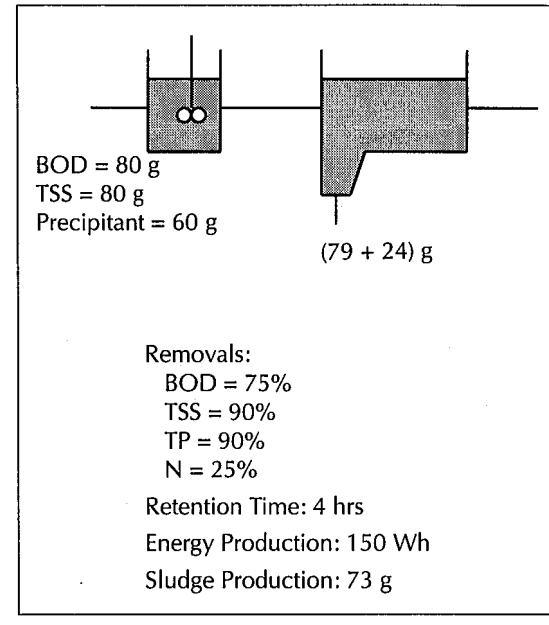

FIGURE 6. Direct precipitation sludge and energy production.

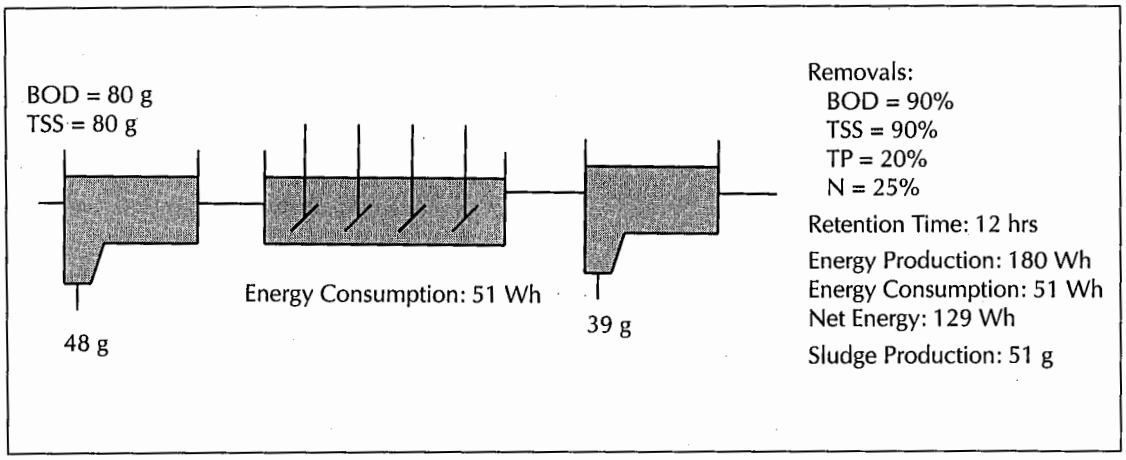

FIGURE 7. Conventional primary treatment followed by an activated sludge process (sludge and energy production/consumption).

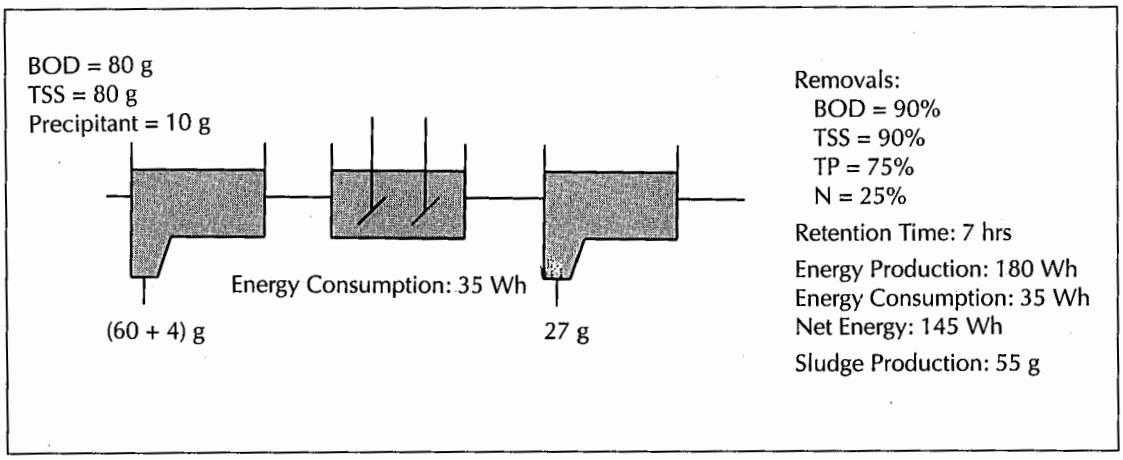

FIGURE 8. CEPT followed by an activated sludge process (sludge and energy production/consumption).

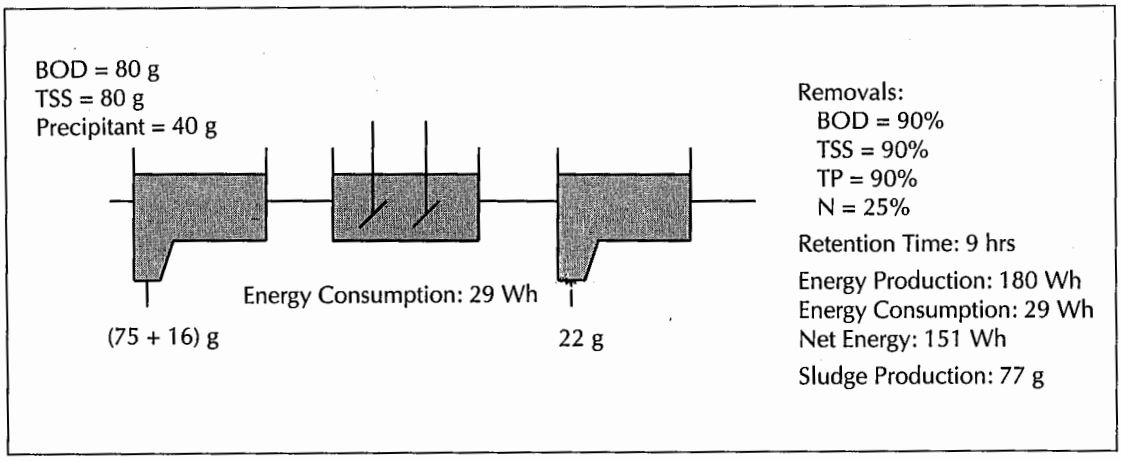

FIGURE 9. Direct precipitation followed by an activated sludge process (sludge and energy production/consumption).

TABLE 5
Characteristics of MWRA Wastewater

Parameter	Influent (mg/l)	Primary Effluent (mg/l)	CEPT Effluent (mg/l)
BOD	171	120	60
SBOD	50	50	50
COD	360	250	120
NH_4-N	10	10	10
NO_3-N	2	2	2
TKN*	19	17	16
Total N	21	19	18
TP	3.5	3.0	1.5

Note: * TKN = Total Kinetic (Non-Inert) Nitrogen

all of these illustrations, typical wastewater having the composition shown in Table 3 is being treated.

To produce one metric ton of a typical chemical used in wastewater treatment requires approximately 200 kWh of energy. This includes all energy requirements (such as for raw material production, heating, transportation of raw material and transportation of the final product to the customer). Therefore, the energy cost associated with chemical usage for CEPT is 3 Wh/pe-d, for direct precipitation is 12 Wh/pe-d, for CEPT and activated sludge treatment is 2 Wh/pe-d and for direct precipitation and activated sludge treatment is 8 Wh/pe-d.

Possible Implementation: The Boston Harbor Project

CEWT may be appropriate for incorporation into the Boston Harbor Project. A main component of the project is the construction of the new Deer Island wastewater treatment plant. The facility is designed to treat an average flow of 480 mgd and a maximum flow of 1,270 mgd. The present plan calls for four primary treatment batteries as well as four secondary treatment batteries. In 1991, the Massachusetts Water Resources Authority (MWRA) contracted with the Massachusetts Institute of Technology (MIT) to evaluate the applicability

TABLE 6
Distribution of Organic Matter in COD Units in MWRA Wastewater

Parameter	Influent		Primary Effluent		CEPT Effluent	
	(mg/l)	(%)	(mg/l)	(%)	(mg/l)	(%)
VFA	36	10	36	14	36	30
Soluble Matter	36	10	36	14	36	30
Colloidal Matter	72	20	72	29	12	10
Suspended Matter	180	50	70	29	0	0
Inert Matter	36	10	36	14	36	30
Total	360	100	250	100	120	100

TABLE 7
Denitrification Capacity and Rate for MWRA Wastewater

	Maximum Removal of Total N (mg/l)	Denitrification Rate (g N/kg VSS-h)	Volume Required for Dentrification	Volume Required for Nitrification
Influent	24	0.7	1.0	1.0
Primary Effluent	20	0.88	0.8	0.7
CEPT Effluent	13	1.4	0.5	0.3

Note: VSS-h = Volatile Suspended Solids per Hour

of the CEPT process for the new plant. That work confirmed the benefits of CEPT as outlined below.[5] The MWRA is currently evaluating CEPT in a 2-mgd pilot facility on Deer Island.

Based on the characteristics of the wastewater that would be treated at Deer Island, the removal efficiencies of different parameters were calculated for both CEPT and conventional primary treatment (see Table 5).

Currently, the MWRA has no requirements for nitrogen removal. However, if nutrient removal may be required in the future, it would be important to determine how the MWRA may react since there are critical space limitations on Deer Island.

The organic matter in wastewater can be divided into five different groups (see Table 6). The fractionation of these groups is important in biological processes because they affect sludge production and biological activity in different ways. For biological nitrogen removal, the volatile fatty acids (VFA) have the highest denitrification rate. The soluble part without VFA is slower. The colloidal and suspended fractions are even slower because they must be hydrolyzed into a more easily degradable form first. As seen in Table 6, CEPT effluent has a higher percentage of easily degradable organic matter.

Assuming 6 mg of COD is needed to remove 1 mg of nitrogen, and that 100 percent of the VFA, 100 percent of the soluble matter, 50 percent of the colloidal matter and 20 percent of the suspended matter will be utilized, and that the denitrification rate (grams of Nitrogen per kilogram of volatile suspended solids per hour) for VFA is 3, for soluble matter is 1.5, for colloidal matter is 0.5 and for suspended solids is 0.3, the denitrification rate and volumes (compared to no treatment) for nitrification and denitrification were determined for CEPT and conventional primary treatment effluents.

The CEPT process requires a much smaller volume than the conventional primary treatment process (see Table 7), although only 13 mg/l of the total nitrogen (18 mg/l) for CEPT effluent can be removed without the addition of an additional carbon source. Hence, any volume freed by CEPT during the secondary treatment process could be used for additional nutrient removal if needed at Deer Island.

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NOTE — *Kemira Kemi AB has developed the use of CEPT to accelerate nitrification and for hydrolysis of the sludge to accelerate the denitrification process, calling the technology HYPRO.*

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Tips for Slurry Wall Structural Design

Construction techniques and site-specific subsurface conditions affect slurry wall performance more than structural details or code interpretations.

CAMILLE H. BECHARA

Reinforced concrete diaphragm walls, commonly known as slurry walls, have been used in Europe since the 1950s and have gained increased popularity more recently in the United States, particularly in major urban projects, including Boston and Chicago.

In the Boston area alone, involvement in the design and construction of slurry walls has been extensive. Besides their use on the Central Artery/Third Harbor Tunnel project and many transit subway projects, several underground structures have been constructed using slurry walls, including segments of the Red Line tunnel extension from Harvard to Alewife stations, Rowes Wharf, 75 State Street, 150 Summer Street, Post Office Square garage, North Station garage and Beth Israel garage (under construction). In all of these projects, the slurry wall acted as the temporary as well as the permanent structural wall. The walls were designed to resist lateral earth/water pressures, earthquake forces and, at the same time, had to be load bearing by supporting column loads from air-rights developments and the below-ground floor reactions. Slurry walls are capable of behaving structurally as temporary and permanent earth/water retention systems, as well as behaving as load-bearing elements (LBEs) with the ability to provide a finished concrete surface. These multiple characteristics of slurry walls have resulted in many advantages over conventional foundation design, particularly when used in congested built-up areas.

Simply put, the construction of a slurry wall panel consists of:

- Excavating a trench approximately two to three feet wide by six to 25 feet long with a grab bucket down to till or rock;
- Filling the trench with a liquid slurry bentonite (primarily clay mineral) as the excavation proceeds;
- Lifting a reinforcing cage (that is assembled at the site) with a crane and placing it in the completed trench under slurry; and,
- Lowering tremie pipes to pump concrete from the bottom up, displacing the slurry liquid as concreting takes place.

Panels can be cast either alternately or successively, and the joints between adjacent panels are intended to be watertight.

Since a slurry wall is typically buried in soil for at least half its face and for the entire service

life of the structure (a perfect soil-structure interaction problem), its design is greatly dependent on the method of analysis used and requires close communication between structural and geotechnical engineers. Therefore, a brief description of these analysis methods used in design, which have been well presented by Kerr and Tamaro,[1,2] is warranted.

Methods of Analysis

There are four primary methods of analysis for slurry walls:

- Equivalent beam on rigid support (rigid method);
- Beam on elastic foundation (Winkler method);
- Finite element method (FEM); and,
- Limit analysis.

Equivalent Beam on Rigid Support (Rigid Method). In this method, the point of zero pressure is computed below the excavation cut (passive pressure equals active pressure) and acts as a fictitious support. As the excavation proceeds, the wall spans as an elastic beam between rigid supports (provided by bracing members or floor slabs) with the lowest support at the point of zero pressure below the subgrade. While this method may be conservative, it does provide an easy structural solution. However, the computed wall deflections from the elastic analysis do not correlate to predicted ground movements

Beam on Elastic Foundation (Winkler Method). Winkler analysis models the passive side of the wall as a series of "springs" based on the modulus of the subgrade reaction of the soil. Data on the elastic properties of the soil are required in order to obtain the soil modulus. The analysis is conducted using general structural computer software in order to determine moments and shears. This method seems to yield smaller forces than the rigid method and also gives a better indication of wall movement.

Finite Element Method (FEM). This method requires somewhat advanced computer software and modeling procedures. It also requires a more thorough knowledge of the soil properties that are modeled in the program and with the construction sequences simulated in the finite element analysis. Its greatest benefit is better prediction of soil movement. Its use for structural design is limited and reliance on other methods is necessary.

Limit Analysis (Plastic). Limit, or plastic, analysis is not typical for foundations or below grade structures. The advantages of this method need to be explored more, but without further study at this stage its economical advantages, if any, must be carefully analyzed.

While the rigid method is well known and has been widely used in the past, the Winkler method has been rapidly gaining popularity. The rigid method offers a quick and easy method of analysis and is certainly worthwhile to consider as a first choice or as a preliminary analysis tool due to its simplicity. The Winkler method offers a more attractive solution in that it yields a more realistic representation of wall behavior and allows complete modeling of the wall based on soil springs as well as brace or floor springs (K-values based on stiffness and volume change), and could be used next as a check for design. Selection of the analysis method should be an engineering judgment that should be exercised on a case-by-case basis, taking into account the degree of conservatism desired.

Certainly, FEM is a powerful analysis tool and offers many advantages during construction/excavation as a means of monitoring wall movements and comparing them with predicted ones. It is recommended that FEM be used as a supplement in instances where adjacent construction may be sensitive to wall movements and better prediction of these movements is required.

Foundation work and underground structures are meant to have considerably longer life than superstructures and, as such, should be analyzed and designed more conservatively since their maintenance, accessibility and repair is much more limited. Given the various unknowns inherent in soil properties, as well as actual versus assumed loads and other such factors, a cautious approach to elastic analysis should be exercised. Regardless of the method that is employed, it is essential that close coordination and communication take place early in the design process between geotechnical and structural engineers, and that a consensus and understanding be reached. This process must be followed by coordination meetings with the

contractor in order to avoid problems during construction. However, the contractor, excavator, inspector, rebar supplier, ironworker and slurry wall contractor should not all be expected to coordinate with each other on a specific shop drawing item. Therefore, the engineer should be the mediating influence between all parties involved.

Slurry Wall Penetration

The depth of wall penetration below subgrade or dredge level is a stability problem. In most cases, it is controlled when the excavation is at its final stage and just prior to installing the lowest slab or bracing member. It is assumed, for economical reasons or soil conditions, that the penetration depth below this level is insufficient to produce restraint at the bottom of the wall (analogy to the free earth support method). Since the wall is free to rotate about its lower end, a factor of safety with respect to passive resistance must be used in the design. A value of 1.50 is recommended as a minimum.

The required penetration, d, is determined by equating to zero the sum of the moments on the active and passive sides about the brace point or floor level above (see Figure 1). This condition yields a cubic equation in which d is easily solved by substituting trial values. The embedment is often arbitrarily increased by 20 percent (an additional two feet for every 10 feet of embedment) to guard against excess excavation or the presence of pockets of weak soil.

Some engineers logically may want to take advantage of the additional bending resistance provided by the wall at the brace elevation and/or the moment capacity of the brace point at the connection to the wall. This approach is certainly an engineering judgment that should be used on a case-by-case basis, depending on the degree of protection provided by the other systems utilized. It is a function of several factors including the overall degree of conservatism in selecting soil parameters, method of analysis, accuracy of loads and groundwater pressure, existing adjacent foundations, etc. The same rationale also should be used in determining the magnitude and applicability of the safety factors that must be incorporated in the design.

Another type of analysis that may be of interest is to consider the effect of base friction on the wall penetration, taking into account the coefficient of friction between the concrete and the rock or bearing material, and then making a determination on whether to rely on it or not.

State-of-the-Art Technology

A few of the projects located in the Boston area, examined from a design criteria standpoint, illustrate some of the current methodology used in the design of slurry walls.

North Station Underground Garage. This six-level underground garage is located at the site of the new Shawmut Center sports arena behind the existing Boston Garden building. Construction is nearly complete, and consists of three-foot thick reinforced concrete slurry walls, 60 to 80 feet deep, embedded in bedrock and with wall panels varying in length from 8.5 to 25 feet.

Due to the large footprint of the garage, the scheduled arena construction above it and the extremely tight physical constraints of the site that is surrounded by the existing Boston Garden building, the Orange Line subway tunnel, the Federal Administration Building, and a Central Artery ramp structure, the top-down method of construction was chosen. Under this method, the slurry walls were built first all around the site, acting as a water cut-off as well as the permanent wall of the structure. Interior steel columns were then installed from grade using a slurry trench technique. The base was filled with tremie concrete, forming supporting caissons or LBEs. The ground floor level was then built around the columns that supported it, and the completed slab braced the slurry wall while excavation was carried out in a mining-like operation, thus eliminating the need for conventional internal bracing or tiebacks. This step was repeated for the lower levels.

Since the slurry wall served as both the temporary and permanent excavation support element for the garage, it was analyzed for all stages of construction, and its design and detailing were comprehensively included in the contract documents.

During the construction process, active lateral pressures were assumed and a triangular pressure distribution was used. The passive pressure was computed at various excavation levels and was developed based on the point of

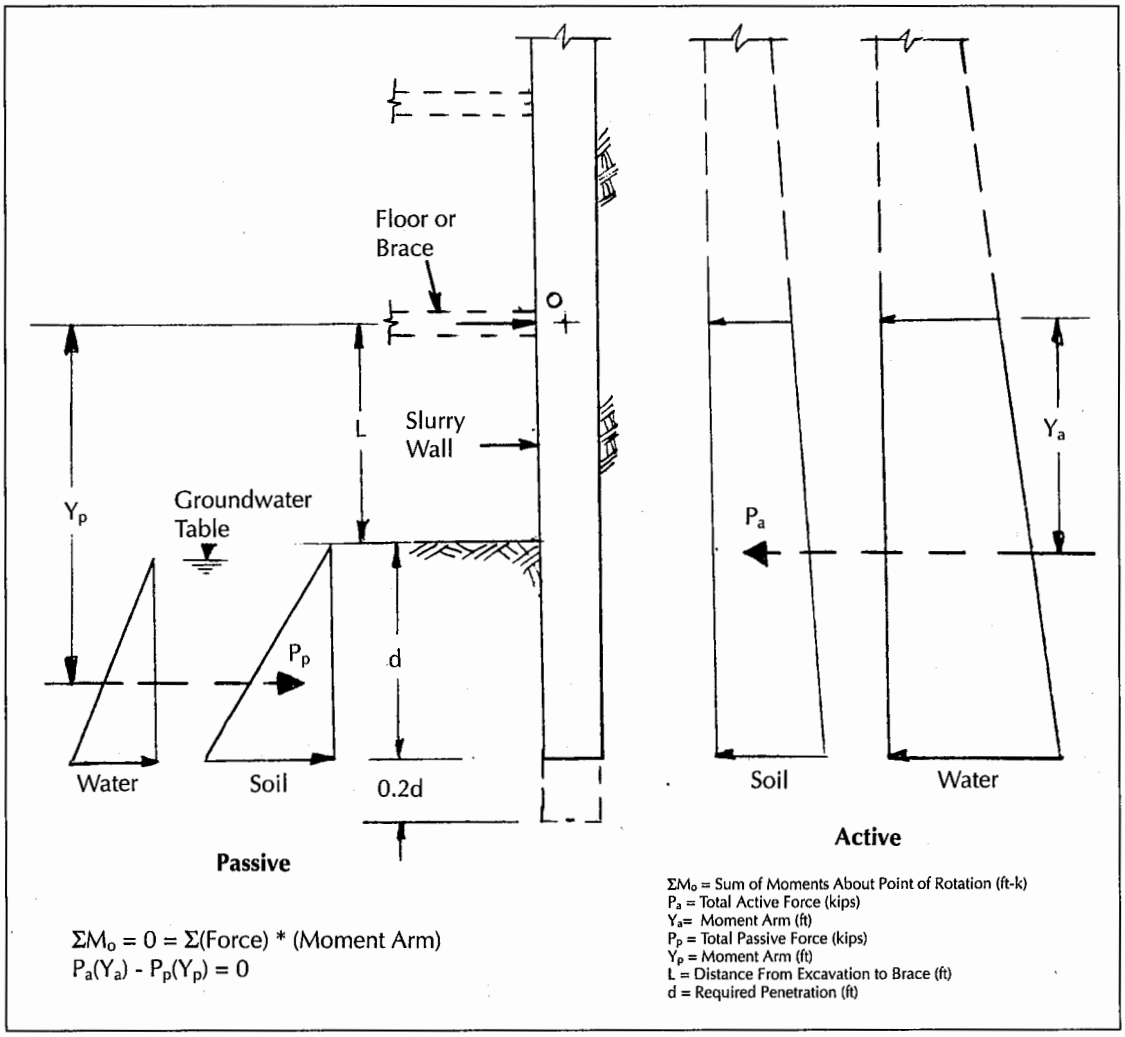

FIGURE 1. Slurry wall penetration.

"zero pressure" needed to compute the reaction below the excavation level (rigid method). This analysis was later checked using the Winkler method by inputting the soil on the passive side as a series of spring constants based on the soil modulus of the subgrade reaction. A finite element analysis was also performed and the wall was analyzed assuming the predicted slurry wall deformations. The calculated bending moments obtained using FEM were lower than the ones from the other analyses.

As construction neared completion, prior to casting the lowest base slab (six levels down), active lateral pressures were also assumed but based on a rectangular distribution. This analysis was used to determine the slurry wall penetration and stability that was obtained by equating to zero the active and passive moments about the level above.

At rest, lateral pressures (triangular distribution) were used for the design of the slurry wall in order to evaluate long-term (permanent) loading.

In all of the above stages, the slurry wall was designed for soil, groundwater and surcharge pressures, and in the long-term stage for seismic pressures. A factor of safety of 1.50 was applied to passive soil pressures for all stages. To obtain a comprehensive bending moment envelope along the height of the wall, the support reactions provided by the floor slabs at

each stage were considered to be rigid supports (zero deformation) as well as flexible springs, taking into account the elastic shortening of the slab as well as volume change deformations resulting from creep, shrinkage and temperature variations.

Computations were performed using the ultimate strength design method (the American Concrete Institute's Building Code Requirements for Reinforced Concrete [ACI 318]).[3] The basis for using the 1.50 safety factor that was applied to the passive pressure in all stages was to minimize wall movement next to adjacent structures and limit cracking (since the wall is the temporary and permanent support, and the stresses in the wall were greater during the excavation stages than in the long-term stage).

Post Office Square Underground Garage. The seven-level Post Office Square garage in the heart of downtown Boston utilized three-foot thick slurry walls approximately 90 feet deep, and the top-down construction method. Slurry wall analysis and design were somewhat similar to the North Station project. The contract drawings took into account the slurry wall reinforcement and detailing for permanent loading. The reinforcement was checked and modified as required for staged excavation during the construction phase. The project was successfully completed in 1990, and has won several design awards.

Rowes Wharf. Located along the Boston Harbor, this wharf structure employed 2.5-foot thick slurry walls on the waterside and landside that were about 70 feet deep. The up-down construction method was used for this project.

In all of the above projects, an extensive construction monitoring program was instituted that included observation wells, inclinometers and settlement monitoring. This program was necessary in order to monitor potential wall movements as construction excavation progressed, and to check against calculated wall deflections and structural behavior.

Theory Versus Practice

What does all this mean to structural design? Engineers fall back to the ACI 318 concrete building code and design a slurry wall for flexure and shear as if the requirements were no different from those for walls cast above ground. The major difference with slurry wall construction is that it is a deep strip foundation built somewhat in the blind without the convenience inherent with exposed construction (in terms of placing and curing concrete, the control of form dimensions and the placing of reinforcement). For these reasons, a more conservative approach to design and detailing is warranted, including making allowances for anticipated construction tolerances and keeping in mind that the finished product does not always turn out as portrayed by the neat straight lines on the contract drawings (see Figure 2).

While flexural and shear design have a great affect on the eventual performance and appearance of the slurry wall, crack control and wall/soil deflection/displacement must also be considered in a typical design.

Flexural Design

As an illustration of recommended design procedures, consider the example of a three-foot thick wall by one-foot wide strip. Typically, a continuous slurry wall is reinforced in two directions and spans vertically between supports (which consist of bracing members or concrete slabs). Therefore, only minimum horizontal reinforcement is required in accordance with ACI 318-89. Horizontal reinforcement, in this instance, can be calculated thus:

$$
\begin{aligned}
(A_{sh})_{min} &= 0.0025 \times b \times h \\
&= 0.0025 \times 12 \times 36 = 1.08\ in^2/ft \div 2\ each\ face \\
&= 0.54\ in^2/ft
\end{aligned}
$$

where:

b = Width of compression face of wall (in)
h = Thickness of wall (in)
A_{sh} = Area of horizontal steel (in^2)

In this case, use a #7 horizontal bar with 12-inch spacing on each face (0.60 in^2/ft).

With the soldier-pile-tremie-concrete (SPTC) slurry wall system that is being used on parts of the Central Artery/Third Harbor Tunnel project, reinforcement may be omitted on the provision that the flexural, shear, compression and bearing stresses are less than the concrete cracking strength and the permissible stresses allowed by ACI 318.1 (Building Code Requirements for Structural Plain Concrete). In this system, the wall spans horizontally a rela-

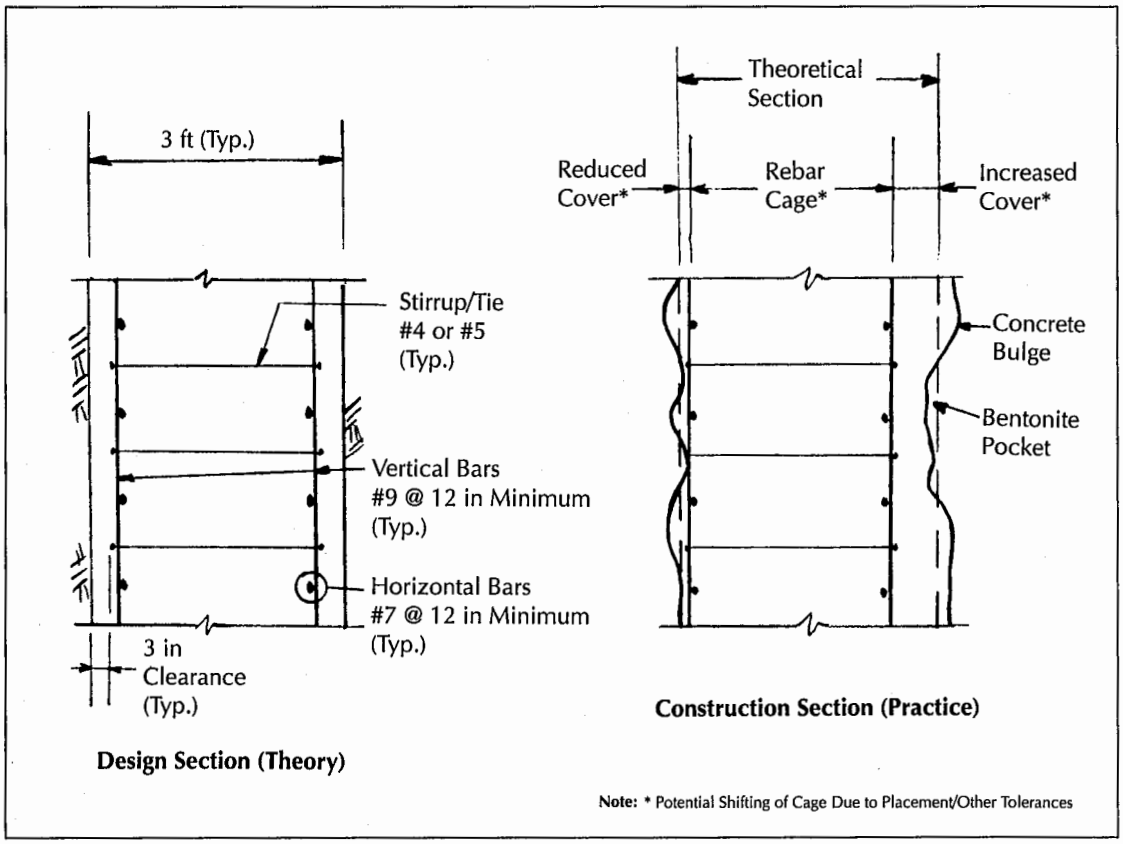

FIGURE 2. Reinforced concrete slurry wall section.

tively short distance (six feet) with a span/depth ratio of less than 2. Thus, the wall behaves as a deep beam member, supported by vertical steel sections that are capable of providing continuous vertical support to resist the overall lateral pressures.

Vertical Reinforcement. As a general rule, use larger bars at larger spacing for ease of concreting and a stiffer cage. Do not use less than six-inch spacing, bundle if necessary and do not splice in congested areas. Consider using #9 and #11 bars at six- and 12-inch spacing and compute the ultimate moment capacity for the various bar sizes and spacing from ACI Commentary R9.1.1:

$$øM_n = 0.9\, A_s f_y\, (d - a/2)$$
$$a = A_s f_y/(0.85 f'_c b)$$

where:

$øM_n$ = Design moment capacity (ft-k)
A_s = Area of tension reinforcement (in^2)
f_y = Specified yield strength of reinforcement (psi)
d = Effective depth (in)
a = Depth of equivalent rectangular stress block (in)
f'_c = Specified concrete compressive strength (psi)

Concrete strength is typically 4,000 pounds per square inch (psi). Concrete strength is greatly affected by any change in the water-cement ratio. This change could occur if the bentonite slurry is intermixed with the concrete, which would increase that ratio and theoretically reduce the concrete strength. However, a good slurry mix should prevent that.

On recent projects, there has been no reason to conduct test cores on slurry walls. Xanthakos indicated that core tests obtained from walls several weeks after concrete placement showed strengths higher by as much as 1,000 psi than the design strength.[4] It would be interesting to

obtain test cores several months — or longer — after excavation has been completed and the slurry wall surface has been exposed.

For concrete strength selection, it may be good practice to specify 4,500 psi or to design for 3,500 psi. This choice is a judgment call. It does not affect the amount of flexural reinforcement, but it may affect shear design and deflection. For a small incremental cost (if there is any at all), it is worthwhile doing, depending on the type of structure involved and until further tests indicate otherwise.

Grade 60 steel is the commonly used steel in reinforced concrete practice. Therefore, the yield strength of reinforcement, f_y, is usually 60,000 psi.

The effective depth of the slurry wall requires some attention since it greatly affects the amount of reinforcing required. ACI 318-89, Section 7.7.1(a), prescribes that "concrete cast against and permanently exposed to earth" have a minimum concrete cover of three inches needed for reinforcement. Furthermore, a tolerance of one inch in placing the steel is prescribed by ACI 117-70 (Tolerances), Section 2.2 (Reinforcement Placement), which is intended for formed walls.[5]

In addition, the out-of-plumb tolerance of the wall should be considered and typically should not exceed one inch from floor to floor. Finally, bulges and cavities occur typically as a result of slurry wall construction, resulting in the loss of cross section and, at times, exposed reinforcement.

Taking all the above factors into account, slurry wall effective depth can be computed as:

$$d = 36 - 3(cover) - 1(rebar\ tolerance) - 1\ (other\ tolerances) - 1/2(\#4\ stirrup) - d_b/2(rebar)$$
$$= 36 - 5.5 - d_b/2$$

For #9 bars with a bar diameter, d_b, equal to 1.13, the effective depth equals 29.9 inches. For #11 bars with d_b equal to 1.41, the effective depth equals 29.8 inches. Therefore, a value of 29.5 inches for the effective depth is recommended. Some may argue that this value is too conservative and it very well may be. However, its adoption is an engineering judgment that should be used on a case-by-case basis and depends largely on the degree of protection required and the type of structure under study.

TABLE 1
Moment Capacities

Bar Size	Spacing (in)	A_s (in^2)	a (in)	ϕM_n (ft-k)
#9	6	2.0	2.94	275
#11	6	3.12	4.59	417
#9	12	1.0	1.47	129
#11	12	1.56	2.29	199

The moment capacities can be computed as shown in Table 1.

One easy method of selecting the required reinforcing is to plot the maximum positive and negative moments at each stage of construction along the full height of the wall and selecting the appropriate reinforcement by comparing the applied moments and the moment capacities.

Minimum Vertical Reinforcement. This requirement may also be subject to debate. One can rightfully turn to ACI 318 Chapter 14 (on walls) and determine the minimum area of vertical reinforcement (in^2) from the following equation in Section 14.3.2:

$$(A_v)_{min} = 0.0015 \times 12 \times 36 = 0.65\ in^2 \div 2\ each\ face$$
$$= 0.33\ in^2/ft$$

In this case, the minimum vertical reinforcement would be #5 bars at 11-inch spacing or #6 bars at 16-inch spacing.

However, bearing in mind that the intent of ACI's requirements for walls is that they serve as primarily compression members, while a slurry wall is primarily a flexural member. Therefore, it is recommended to use ACI Section 10.5.2, which calls for providing reinforcement at least one-third greater than that *required by analysis.* What is required by analysis could be, as a minimum, the moment capacity of the wall as a plain concrete section, or cracking moment, determined from the modulus of rupture of concrete, f_r, as shown in Section 9.5.2.3:

$$M_{cr} = f_r\, I_g/y_t$$

where:

f_r = Modulus of rupture (psi) = $7.5\sqrt{f'c}$ = 474 psi

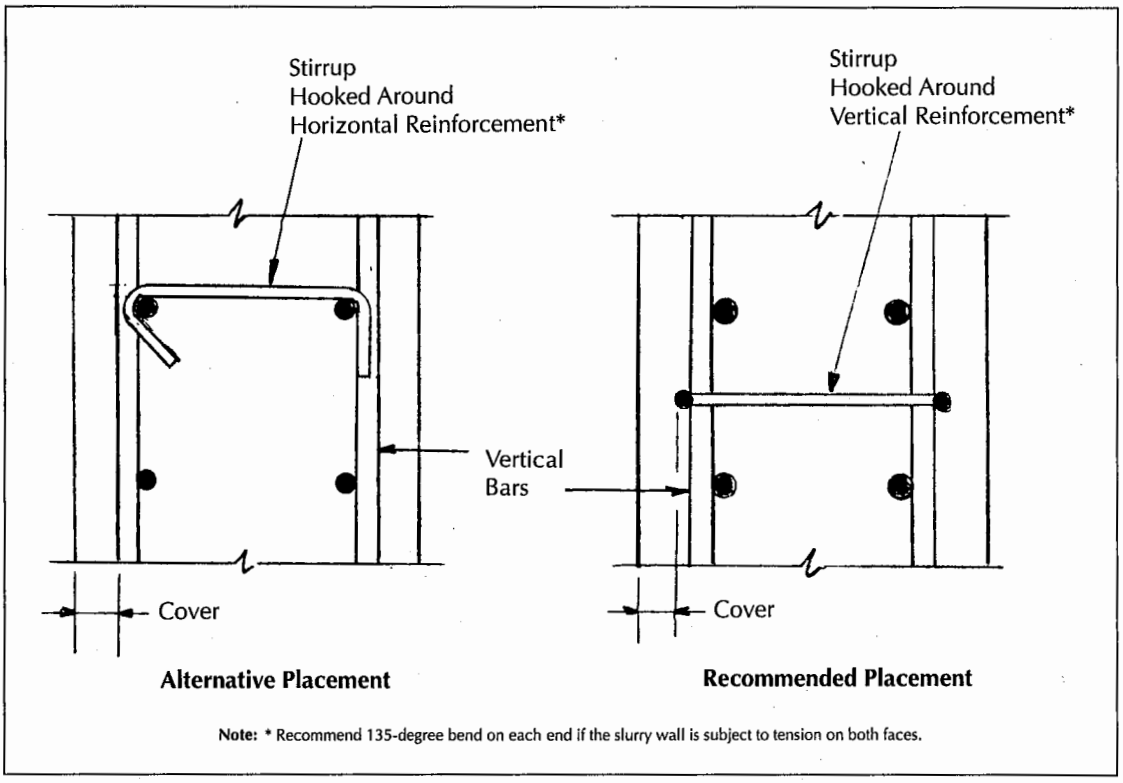

FIGURE 3. A section of the stirrup reinforcing detail.

I_g = Moment of inertia (in^4) = $bh^3/12$ = 46,656 in^4

y_t = Distance from neutral axis to extreme fiber in tension (in) = $h/2$ = 18 in

The wall's cracking moment is equal to 102.4 ft-k. The bonding resistance coefficient, K_u, is equal to $(M_{cr}\times12{,}000)/(bd^2)$ or $(102.4\times12{,}000)/(12\times29.5^2)$, which yields 118 psi. The ratio of tension reinforcement, ρ, is equal to A_s/bd, which yields a value of 0.0022 (from the ACI Design Handbook Table 2.2, Flexure). ρ_{min} is equal to 0.0022×(4/3), or 0.0029. The minimum area requiring tension reinforcement, $(A_s)_{min}$, is equal to 0.0029×12×29.5 or 1.03 in^2/ft, in which case use a #9 bar at 12-inch spacing (1.0 in^2/ft).

Therefore, if the applied moment is less or equal than 102.4 ft-k, provide minimum reinforcement as computed above along each face of the wall. It is also important to point out that in completing a project's detailed drawings, the total number of bars provided in the wall should be based on the overall panel length. This requirement is because of the large concrete cover to the vertical bars from the ends of the panel, which can be as much as nine inches and is dictated by the size of the end stops.

On the other hand, some may consider ACI Section 10.5.3 more appropriate for a slurry wall since it is essentially a one-way structural slab of uniform thickness. This assumption is true from the standpoint of the width to depth ratio. However, a slab is typically much thinner than a three-foot slurry wall section. Using Section 10.5.3, or $(A_s)_{min}$ = 0.0018×12×36 = 0.78 in^2, will result in a moment capacity of 101 ft-k, which is slightly smaller than the cracking strength of plain concrete (the 102.4 ft-k above). Therefore, it would not be effective or contribute to the wall's bending resistance. Moreover, a slab's weight is not normally subjected to a true permanent and uniform loading. That is the reason why the minimum reinforcement required for slabs is less than that for beams, since an overload as discussed in the ACI Commentary would be distributed laterally and a sudden failure would be less likely. A slurry

wall, on the other hand, is permanently subjected to the uniform loading of soil and water (primary loads) and, hence, for design purposes it is recommended to use Section 10.5.2.

Shear Design

With regard to flexure, it is recommended to plot the shear force envelope, V_u, along the height of the wall and compare it with the shear capacities, $øV_n$. The ultimate shear capacity of the concrete is determined by ACI Section 11.3 as follows:

$$øV_c = (0.85 \times 2 \times \sqrt{4{,}000} \times 12 \times 29.5)/1{,}000 = 38 \text{ kips/ft}$$

If the shear force, V_u, is less than 38 kips/foot, then no shear reinforcement is required. If V_u is greater than 38 kips/foot, provide minimum stirrups to meet ACI Section 11.5.5.3. Use 18 inches for the horizontal spacing, b_w. For the recommended vertical spacing, S, first calculate the maximum vertical spacing, S_{max}, which is equal to $d/2$ (29.5/2 or 14.75 inches). Therefore, it is recommended to use 12-inch vertical spacing.

The minimum vertical reinforcement required is determined by:

$$(A_v)_{min} = 50 b_w S/f_y = (50 \times 18 \times 12)/60{,}000 = 0.18 \text{ in}^2/\text{ft}$$

In this case, use a #4 single leg stirrup at 18-inch horizontal and 12-inch vertical spacing. Shear capacity for this configuration is calculated as:

$$V_s = A_v f_y d/S = (0.2 \times 60 \times 29.5)/12 = 29 \text{ kips/ft}$$
$$øV_n = øV_c + øV_s = 38 + (0.85 \times 29) = 63 \text{ kips/ft}$$

If the shear force is less than or equal to 38 kips/ft, then no shear reinforcement is required. If the shear force is greater than 38 kips/ft and less than or equal to 63 kips/ft, use a #4 single leg stirrup at 18-inch horizontal and 12-inch vertical spacing. Shear capacities for different stirrup spacings and sizes can be developed in a similar manner.

In contrast to flexural design, the shear design in this case is quite straightforward. However, some disagreement lies in the detailing and placing of the stirrups. The issue is whether to hook the shear reinforcement around the vertical bars (recommended) or around the horizontal bars (see Figure 3). The recommendation is based on the interpretation of the following ACI code sections and additional comments, and is intended to promote further discussion.

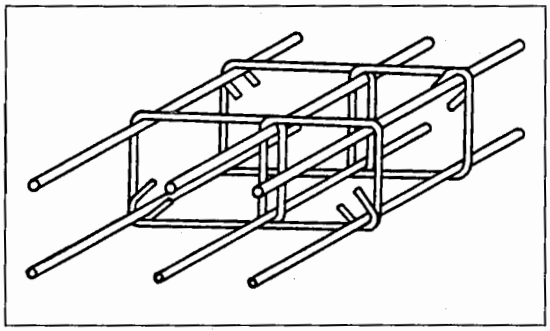

FIGURE 4. Slab stirrups.

ACI 318-89 requirements for anchorage (Sections 12.13 and R12.13: Development of Web Reinforcement) state that web reinforcement or stirrups shall be carried as close to the compression and tension surfaces of a member as cover requirements and the proximity of other reinforcement will permit because, near ultimate load, the flexural tension cracks penetrate deeply. Section 12.13.2 states, in part, that #5 bar stirrups (as well as smaller stirrups) should be anchored with a standard hook around *longitudinal reinforcement*. The vertical bars in the slurry wall are the flexural reinforcement and act as the longitudinal reinforcement. Therefore, the stirrups should be hooked around this reinforcement. It should also be noted that ACI anchorage requirements for stirrups have been developed primarily for slabs and beams in which the longitudinal (or flexural) reinforcement is enclosed by the stirrups. A wall subjected primarily to bending is, for design purposes, a vertical slab. The ACI code also addresses this requirement in the following sections:

> Section 7.11.2 states that "lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement." This requirement is applicable for earthquake design where the dynamic or cyclic reversal of stress occurs. It is intended for building or superstructure

design in order to guard against excessive cracking caused by tension and to reverse shear developing on the bottom of beams at the supports. Even though stress reversal in slurry wall construction is more gradual (since it is typically associated with staged excavation), these reversals of stresses do cause reverse tension in the wall at different support points and between supports. Since the short-term loading (during excavation) seems to always control design and flexural tension cracks penetrate deeply near ultimate load, hooking the stirrups around the reinforcement is recommended.

Commentary on Section R11.12.3 (Special Provisions for Slabs and Footings) states that "research has shown that shear reinforcement consisting of bars or wires can be used in slabs provided that it is well anchored." The anchorage detail used in the tests is shown in Figure 4.

Section 14.3.6 requires that vertical reinforcement be enclosed by lateral ties if the vertical reinforcement area is greater than 0.01 times the gross concrete area, and where vertical reinforcement is required as compression reinforcement. This situation is particularly applicable if the slurry wall is supporting large column loads.

Conclusion

In summary, flexural and shear design for slurry walls must be approached not only from structural integrity and code requirement viewpoints, but must also take into account — during the design and detailing of the contract documents — the construction practices in the area and the increased tolerances associated with this type of construction. The recommendations presented here are intended to facilitate structural design, bring awareness to the differences between design and construction, and provide a more durable concrete wall that will exhibit fewer cracks and leaks, and help reduce any long-term maintenance that may be required. The ultimate performance of the slurry wall is more significantly affected by construction techniques and site-specific subsurface conditions than it is by structural details and code interpretations.

NOTE — *The author's extensive involvement in the North Station project rendered that description in the section on State-of-the-Art Technology to be more elaborate than the other noteworthy projects.*

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The Effectiveness of Municipal Wastewater Treatment

Past and present federal policy on pollution enforcement continues to have significant impacts on the effectiveness of wastewater treatment at the local level.

HOLLY JUNE STIEFEL

In February 1991, the United States Environmental Protection Agency (EPA) released a report that claimed that "a strong enforcement program is one of our highest priorities for the Agency" and "we must maintain an enforcement program that will . . . promote pollution prevention."[1] James M. Strock, then Assistant Administrator for Enforcement, stated in his introductory letter that "enforcement is the means by which we assure that the promise of our environmental laws and regulations are realized."

EPA regulators believe a stronger emphasis on enforcement is the most promising way to improve the compliance rates of municipal wastewater treatment plants. Prior to its increased enforcement efforts, the EPA sought to improve performance and ensure compliance through technical support and training assistance combined with financial aid through the municipal wastewater treatment Construction Grants Program (CGP). This program, established as part of the 1972 Federal Water Pollution Control Act (FWPCA), provided approximately $60 billion in grants and loans to help fund municipal wastewater treatment systems. In many ways, the program directly linked compliance enforcement with a community's financial ability to construct or upgrade a plant. Enforcement measures were rarely applied to communities eligible for federal funding, even if the community's plant routinely violated federal standards.[2]

The Water Quality Act of 1987 signaled the end of federal funding for municipal plants because direct federal grants to communities were phased out and replaced with loans from State Revolving Funds (SRFs). While the federal government has provided initial capitalization grants to help establish SRFs, all federal assistance under the Clean Water Act (CWA) will end in 1994 since the CWA has not been renewed. Even though temporary provisions have been made to continue to fund SRFs until the CWA is reauthorized, states and municipalities will share increased responsibility for funding wastewater treatment plants, and the lack of federal funding will no longer be an acceptable excuse for noncompliance.

Funds available through SRFs are only a small fraction of funds needed to upgrade municipal plants. EPA's 1992 Needs Survey estimated that $137.1 billion would be needed for wastewater treatment over the next 20 years.[3] Partly in response to the lack of funding, the EPA established the Public-Private Partnerships Initiative. Currently known as Partners Rebuilding America, this program was created primarily to develop "innovative financing approaches" to construct or maintain plants without federal funding.[4] Through this program the EPA intended to increase private-sector involvement in wastewater treatment from financing to ownership.[5] Although public-private partnerships or privatization have been viewed by the EPA predominantly as an option to relieve the financial burdens of municipal plants, the agency also recognized that privatization can reduce costs, speed project construction, guarantee proper performance, and preserve jobs.[6]

Current debate indicates that Congress and the EPA may not turn over wastewater treatment responsibility to the states and local communities in 1994 as currently planned. In 1991, Senator Max Baucus proposed federal legislation that called for a partial return to the grant system. Currently, several bills propose to extend federal funding of SRFs beyond 1994. Others seek to allocate a portion of federal funding for direct grants to specific projects or encourage suspension of the 20 percent state matching grant requirements. Extension of federal capitalization grants, conversion of loans to grants, provision of grants to specific communities and elimination of state matching funds all encourage communities to view wastewater treatment as a federal responsibility and discourage communities from discovering other ways to meet their treatment needs.

The performance of a municipal wastewater treatment plant can be evaluated in a number of ways. Typically, a plant is judged to be performing satisfactorily if the quality of its treated effluent meets the requirements of its National Pollution Discharge Elimination System (NPDES) permit. Overall compliance as reported by the EPA and responsible state agencies normally refers to the percentage of a group of treatment plants that satisfy their NPDES permits. National water pollution control policy, as reflected by the FWPCA and in the NPDES permits, has received serious criticism and is a matter of ongoing public debate.

Municipal Wastewater Treatment Plant Compliance

Compliance History. The 1972 FWPCA and its amendments provide the basis for the current policies regulating municipal wastewater treatment plants (MWTPs). This act represented the first attempt by the federal government to take responsibility for establishing water quality goals and enforcing regulations and also to establish plans to achieve those goals. The act's two main goals were to achieve fishable and swimmable water by 1983 and to eliminate all discharges of pollutants into navigable waters by 1985.

In support of these goals, the act established the NPDES. Compliance with the NPDES is routinely used as one way to evaluate plant performance. This system requires point-source dischargers to obtain a permit before commencing any discharge. Each permit specifies the type and amount of pollutants allowed as well as a time schedule by which compliance with the limitations is required. For regulatory purposes, the permit also specifies sampling methods and schedules as well as requirements that outline the obligation to report collected data to the appropriate regulating authority.

Before the passage of the FWPCA, effluent limitations were set by individual states. The FWPCA required effluent limits to be based on what was technologically achievable rather than what any state might judge most appropriate for its individual water courses and effluent discharge points. Thus, all MWTPs were required to meet effluent standards achievable through the use of best practicable waste treatment technology. Although the descriptive technological titles and the corresponding levels of treatment have changed, these particular effluent limitations have always been linked to a specific technological capability.

The FWPCA required municipal dischargers to meet secondary treatment guidelines by July 1, 1977, as well as to install best practicable water treatment technology by July 1, 1983. Soon after the act was passed, the EPA esti-

mated that 50 percent of existing municipal dischargers would not be able to meet the secondary treatment guidelines by 1977. Based on data from the 1976 Needs Survey, the Water Pollution Control Federation (since renamed the Water Environment Federation) estimated that only 33 percent of municipal dischargers would be able to meet the secondary treatment deadline.[7] As late as September 1978, the EPA stated that "[a]n analysis of compliance indicates that the majority of publicly owned treatment works (POTWs) have not completed construction necessary to meet the 1977 treatment requirements."[8]

In 1977, amendments to the FWPCA were passed and the act was renamed the Clean Water Act. This act changed several of the deadlines facing municipal treatment plants. Any plant that discharged into marine waters could apply for a waiver of the secondary treatment guidelines if the applicant could show that the discharge would not harm the ocean environment. An extension of the 1977 deadline until 1983 was available to plants that lacked adequate federal funding to complete construction by the 1977 deadline. The 1981 Construction Grants Amendments extended this deadline to July 1, 1988.

In December 1983, a United States General Accounting Office (GAO) report to the EPA Administrator stated that "noncompliance with permit limits was widespread, frequent, and significant."[9] The GAO randomly selected 531 major dischargers (municipal and industrial) and found that:[9]

> *[M]unicipal dischargers exceeded their permit limits more frequently than industrial dischargers. For example, about 59 percent of the municipals exceeded their concentration limits for more than six months while only 33 percent of industrials exceeded those limits for more than six months in the 18-month period reviewed.*

Of the dischargers experiencing significant noncompliance (defined as exceeding permit limits for one or more pollutants by 50 percent or more for at least four consecutive months), 69 percent were municipal.

By the time National Municipal Policy enforcement rules took effect in 1985, more than 60 percent of the nation's major municipal plants were meeting effluent limitations. Of the remaining plants that needed upgrading, 70 percent achieved compliance by the July 1, 1988, deadline. About 30 percent of the more than 400 remaining plants were considered technically in compliance because they were subject to legally enforceable compliance schedules.[10]

TABLE 1
Secondary Treatment Compliance as of July 1, 1988

	Total*	In Compliance
Major POWTs**	3,731	88%
Minor POWTs***	11,755	85%

Notes: * Includes all plants except for those whose operational discharge data were not confirmed and those who were not in compliance but expected to be by 9/30/88. ** Greater than 10,000 population or 1 milion gallons per day (mgd). *** Less than 10,000 population or 1 mgd. Data from Ref. 11.

The EPA estimated that 88 percent of the major municipal dischargers were in compliance by the July 1, 1988, deadline for secondary treatment — a significant increase in just three years.[11] Table 1 shows that 85 percent of minor municipal dischargers met the same goal. However, not all dischargers were included in the EPA's study.

The EPA stated that, as of January 1990, "89 percent of all major municipals . . . [had] completed construction to meet final effluent limits."[1] This statement did not include plants that were experiencing difficulties with operation and maintenance or other problems that prevented compliance but did not require construction to improve performance. In addition, it did not include those plants that were subject to interim limits as part of an enforceable compliance strategy designed to meet final limits.

As late as 1992, the GAO reported that compliance problems had not been fully documented since the EPA's compliance monitoring was "limited to major and significant minor wastewater treatment facilities."[12] Twenty percent of states believe noncompliance will increase, especially as new wastewater regulations such as stormwater permits and toxic discharge limits are enforced. Utah has expressed concern that its state health depart-

ment may be forced to condemn entire towns unable to afford wastewater plant upgrades.

Interpretation of Compliance Figures. Several limitations impact evaluations of the significance of the above compliance figures. Significant noncompliance indicates that these plants are routinely discharging prohibited effluent rather than experiencing a one-time problem. Although two plants can receive the same volume of inflow and both can be defined as not complying, it is possible for one to experience a greater number of individual violations, and thus treat a larger amount of water in a prohibited manner. Also, compliance figures are based on individual plants rather than the quantity of water treated. It cannot be determined what percentage of water is not meeting permit limitations. Furthermore, compliance, as defined by the EPA, only refers to plants that have permits and, therefore, are able to comply or not comply with permit limits. If all the plants that needed permits had permits, noncompliance figures might be significantly higher.

In 1984, the EPA permit backlog was estimated at 16,062 applications, most of which had been on a waiting list since 1982. Over 98 percent of the applications were for minor municipal plants.[13] Because of inadequate funding, staff shortages and the complexity of the permit process, the EPA could not even *identify* all the plants needing permits. Even when identified, some EPA regions or states did not attempt to permit minor dischargers.

By the end of fiscal 1992, the nationwide permit backlog accounted for 15 percent of the 583 major and 29 percent of the 3,418 minor municipal plants. If this large number of unpermitted minor dischargers were to receive permits, overall municipal compliance rates might drop since minor dischargers historically have lower compliance rates than major plants.

Finally, a knowledge of the procedures used to monitor, sample and evaluate effluent quality is essential to understand compliance. Compliance rates, even for a well-defined class of plants, do not actually indicate the percentage of plants producing effluent within the permit limitations. There are several reasons why this discrepancy may exist. For example, the existence of a violation may never be revealed to the regulatory agency. This may be because the violation did not coincide with sampling or the discharger did not file the required discharge monitoring report (DMR). The GAO has reported that serious discharge noncompliance could be concealed by the large number of incomplete or missing DMRs.[9]

Even if a plant submits a DMR with the quantity or type of effluent in violation of permit limitations, noncompliance is not always assumed. The same GAO report stated that not all EPA regions or states treated incomplete DMRs as violations, and many laboratories analyzing the discharge as part of the DMR preparation were not able to perform an accurate analysis of the effluent. Through its Discharge Monitoring Report Quality Assurance Program, the EPA submits samples to laboratories analyzing effluent to ensure accuracy. Results from its 1980 and 1982 surveys revealed that 68 percent of municipal samples were analyzed incorrectly for one or more pollutants.[9] Determining compliance is somewhat subjective in that the regulatory agency must determine by what amount the effluent limitation can be exceeded.

Even with these noted limitations, these figures indicate that a significant number of individual plants have experienced difficulty complying with permit requirements. However, it is obvious that the above compliance figures are inadequate to evaluate the effect of compliance on the amount of water treated, or even the general progress toward the goals of the FWPCA. The terms, *compliance, noncompliance* and *significant noncompliance* have a number of different definitions and can be applied to several different requirements established under the FWPCA and its amendments.

Assessment of MWTP Performance

EPA Performance Evaluations. As part of the 1972 FWPCA, the EPA Administrator is charged with conducting an annual survey to compare the efficiency of wastewater treatment plants constructed with federal grant funding with the efficiency planned for the plant before it was built. Four years after the act's passage, Walter G. Gilbert, chief of the EPA's Municipal Operations Branch in the Office of Water, used these annual surveys as a database to deter-

mine the relation between operation and maintenance (O&M) and MWTP performance. These surveys, combined with data from an EPA O&M program started before the passage of the FWPCA, provided data needed to evaluate 1,517 municipal plants.

To determine if plants were meeting their original design performance objectives, Gilbert focused on five-day biological oxygen demand (BOD) and total suspended solids (TSS) removals. His results indicated that one-third of the plants failed to meet the design BOD removal criteria and one-half failed the design TSS criteria. A similar survey, conducted by the state of Illinois, also indicated that plants built with federal grant money were not meeting design criteria.[14]

Gilbert also examined the ability of the secondary treatment plants to meet secondary treatment standards as defined by the FWPCA under the CWA. These standards require at least an 85 percent removal of both BOD and TSS, and stipulate 30 mg/L as the maximum allowable concentration of either parameter. Although Gilbert's study concluded that less than 50 percent of the secondary plants were meeting design criteria, many of the plants were in operation before this level of treatment was established as a goal, and thus not all plants examined were *designed* to meet these parameters. However, the examination did reveal a positive relationship between operational flexibility and compliance. Trickling filter plants had the most trouble meeting the secondary criteria; activated plants performed best.

Gilbert identified several factors influencing the performance of grant-funded plants:[14]

- Plants with detailed maintenance schedules and records consistently performed better than plants with poor records.
- The best performers commonly had an O&M manual that was written specifically for that plant rather than just for that plant's design type.
- Inadequate knowledge of the treatment process led to poor treatment.
- The amount of money spent on training per salary dollar was consistently higher for plants performing above design criteria.
- Excluding plants that were hydraulically or organically overloaded, the most significant factor contributing to poor performance was the operator's inability to utilize fully the facility's design capacity.

In the mid-1970s, the EPA funded a two-part study of municipal treatment plants in response to data released in the 1973 and 1974 editions of the Clean Water Report to Congress.[15] These reports indicated that one-third of MWTPs constructed with federal funding were not meeting design criteria. Initially, two private contractors were hired to identify, quantify and rank the major factors that limit biological wastewater treatment plant performance. Although the focus was limited to treatment problems resulting from what was commonly described as poor O&M, the contractors noted that this category actually included a wide range of factors including staffing, salaries, design, management, budget and traditional maintenance factors.

The results of the study were similar to Gilbert's findings, but much more detailed. The number one performance-limiting factor present, regardless of plant location or treatment type, was that operators were not applying treatment concepts and testing to process control. This factor was present in all but two of the 50 facilities studied and was the leading cause of poor performance at 15 plants. A lack of sufficient understanding of general sewage treatment principles ranked as the second highest contributor to inadequate performance. The third highest factor was a lack of proper technical guidance. Other factors identified included inadequate laboratory procedures, lack of process flexibility and improper design — resulting in many incomplete and marginally operable facilities.[16]

Initial results from the study prompted further efforts to identify performance-limiting factors and develop a corrective plan for improving performance. The plan, the Composite Correction Program (CCP), is still used today to optimize municipal plant performance. Although the additional study still focused on O&M, the results revealed that over 20 percent of the plants studied required major facility design modifications in order to maintain continuous compliance with secondary standards. Since facilities with noticeable design inade-

TABLE 2
CCP Main Recommendations

Operating Personnel

- Improve understanding of treatment process through increased training & operator certification.
- Recognize the extent of problems that can occur & seek assistance in problem solving.
- Accept assistance as a beneficial experience rather than as a reflection of poor ability.

Plant Managers & Municipal Officers

- Recognize the importance of on-site training & well-trained operators.
- Provide the necessary budget for training & adequate salary for plant operators.
- Document the design potential of plants & require that assistance be provided by qualified personnel.

Regulatory Agency

- Expand enforcement activities for plants violating NPDES permits.
- Seek plant performance improvement before resorting to additional construction.
- Improve personnel capability to correct the occurrence of improper technical guidance.
- Focus guidance on the limiting factors identified by the CCP study.

Equipment Suppliers

- Provide flexibility & controllability of equipment.
- Provide realistic O&M requirements.
- Improve technical capability of startup personnel.

Engineering Consultants

- Improve design, especially for CCP-identified design problems.
- Improve training of personnel to avoid the frequent occurrence of improper technical guidance.
- Develop capabilities to implement CCP.

Note: From Ref. 16

quacies were eliminated from consideration when choosing the initial plants, it is clear that a significant number of plants have design deficiencies that are not obvious.

Application of the CCP study to noncomplying plants was successful enough that the consultants recommended its further development and application on a broader scale. A final report, based on the full three and a half year study (including detailed evaluations of five plants), produced a number of recommendations directed at the five groups of people that have significant involvement in the wastewater treatment process (see Table 2).

Performance-Limiting Factors & Corrective Recommendations. Performance-limiting factors identified by Gilbert and the CCP study fell into five areas:

- Problem recognition;
- Knowledge and training;
- Design, construction and technology;
- Finance; and,
- Enforcement.

Failure to recognize problem areas appeared to have significant impact on a plant's ability to comply. The areas that contributed to poor performance could not be addressed until they were recognized. Failure to recognize the problems allowed continued poor performance. Examples that illustrate the problem of recognition include inadequate emphasis placed on proper training and advice, failure to recognize what design/equipment problems are most likely to occur, poor documentation of the potential performance levels to be expected, and ignorance of the potential for improved performance without the need for additional construction.

Inadequate operator application of treatment concepts, insufficient knowledge of general sewage treatment principles, and incorrect technical guidance have limited the ability of operators and owners to run even plants where there are no design or budgetary limitations. Knowledge of treatment concepts and proper training are inadequate at a number of levels. Wastewater treatment technology has advanced significantly since the passage of the 1972 FWPCA and the training of operators, staff, regulatory personnel and consultants is not always adequate to design, construct or maintain treatment plants.

Although plants with obvious design flaws or equipment troubles were excluded from the CCP study, the EPA recognized that:[17]

A major conclusion of this survey was that errors in design were severely limiting the opera-

tor's ability to achieve maximum performance from the facility.

As a result, the EPA published a handbook to help identify and correct design deficiencies in existing plants and prevent the same mistakes from occurring during the design and construction of new plants.

This handbook stated that performance and reliability problems, as well as poor safety practices and decreased flexibility of plant process control, could result from design deficiencies. Design deficiencies included the improper selection of equipment and the inability to make accurate estimates of future plant-operating conditions, including the consistency and amount of inflow expected (a situation that would result in the construction of improperly sized plants that would be difficult to modify or upgrade).

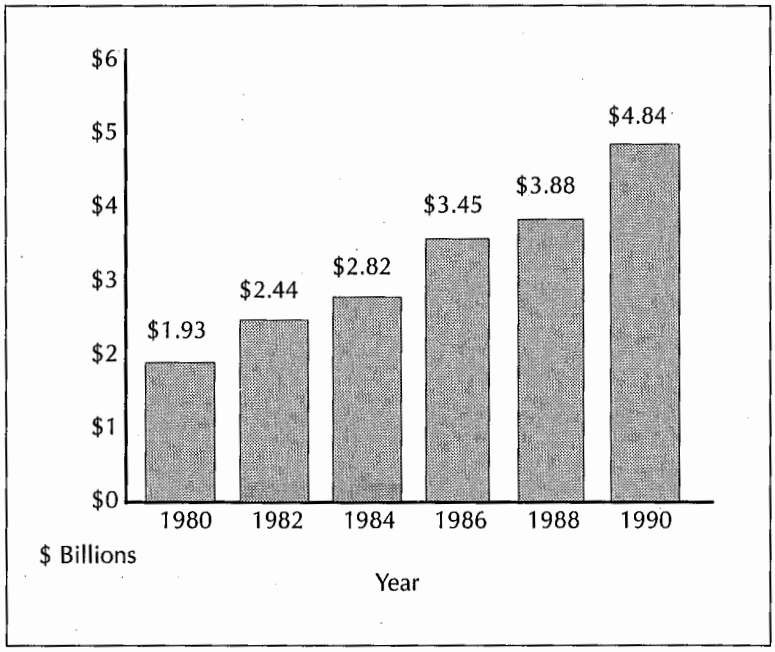

FIGURE 1. Annual O&M expenses for 144 AMSA members.

Despite almost $60 billion granted to plants under the CGP, local plants were still facing budgetary restrictions. While the EPA's 1992 Needs Survey estimated that $137.1 billion would be required to meet wastewater needs for the next 20 years, this estimate did not include O&M costs.[3] The lack of money appropriated for training, salaries and benefits has contributed to poor staff quality and performance. Inadequate expenditures on O&M needs has accelerated plant deterioration and has started a vicious cycle of requiring more O&M expenditures to cope with the physical decline of the plant.

As technology advances, O&M costs will rise and communities must be prepared to face significantly higher costs in the near future. As shown in Figure 1, the Association of Metropolitan Sewerage Agencies (AMSA) predicts that O&M expenses will rise at nine to 11 percent per year — in effect doubling the cost every eight years.

The Effect of Federal Assistance For Municipal Wastewater Treatment on Compliance

The financial ability of a community to construct and operate a wastewater treatment plant directly affected the community's ability to comply with the CWA. In an attempt to equalize the ability of communities to comply with the act, Congress provided enormous amounts of funding for MWTPs through the CGP. Because encouraging compliance was the program's primary goal, penalties for noncompliance were linked to the financial ability of a community to support a properly operating plant. Compliance deadlines were extended and enforcement efforts were withheld from communities waiting to receive a federal grant. In addition to funding, the federal government provided technical assistance to ensure efficient use of federal funds and to promote self-sustaining community O&M programs.

Construction Grants Program. The CGP's effects on performance and compliance must be considered for at least two reasons. First, the CGP can be viewed as a positive influence on compliance in that it provided funding for plants that were experiencing significant com-

pliance problems. The Association of State and Interstate Water Pollution Control Administrators, testifying before a House subcommittee on a February 1989 proposal by then President Bush to cut funding to the CGP, expressed a common belief that municipal noncompliance would escalate with lower funding.

A second reason for considering the structure and influence of the CGP is its impact on local government finance and responsibility. The enormous amounts of CGP funds available to local governments frequently displaced local spending for wastewater treatment plant construction and encouraged communities to view wastewater treatment as the federal government's responsibility. In 1973, when federal grants for wastewater treatment were just beginning, the displacement of local funds was estimated to be 200 percent of the federal grant funds, thus producing a net decrease in funds available for wastewater treatment.[18] In addition, the promise of federal funds specifically targeted to increase compliance provided perverse incentives for communities not to comply (since compliance was rarely enforced and noncompliance actually increased the chance that a local government would receive a grant).

The CGP has allocated about $53 billion in grants and $7 billion in loans for the construction of wastewater treatment plants through individual states.[19] Each state received a portion of the year's allocated funds based on the state's population and projected wastewater needs. The money was allocated through a required state Project Priority List, which ranked projects in order of need.

Initially, states had a tendency to fund projects as soon as they were eligible because unused state money reverted back to the federal government if the money was not used within a specified time period. However, many states found that their share of the federal money was inadequate to meet the state's needs because of stricter funding eligibility restrictions and a limited amount of funding available for each state. As a result, states announced that their share of federal funds would be available on a need basis only. Need was judged primarily on the ability or inability of a plant to comply with regulations. Most states allocated the money to the areas with the worst performance.

Not all municipalities constructing plants during the program's existence received federal funds. However, almost all plants were influenced by the program. Seeing the availability of free federal funds, many plants sought federal funding to upgrade existing plants. Not all of these plants were in need of improved treatment to meet effluent standards. The early period of the program provided federal funding to support reserve capacity that was not needed to comply with the FWPCA. The liberal funding of projects resulted in many communities that, almost twenty years later, still have plants with adequate capacity and low sewer rates.

Local officials desired new plants for a number of reasons. Although the CGP eventually disallowed the funding of reserve capacity, plants were still built with future needs in mind. Plants were typically designed for a 20- to 30-year period. Many communities realized the importance of wastewater treatment capacity in attracting new industry. New industry created jobs, funded other projects and contributed to the overall economic welfare of the community. As a result, the desire to construct plants with large amounts of free federal money commonly outweighed the benefits of operating existing plants in a fashion that would achieve compliance.

Many local officials openly admitted to the pressures placed on them by the public to construct new plants and minimize local costs. If the local officials tried to finance a new plant with local funding by increasing sewer fees, the community would be likely to vote them out of office. This pressure was routinely passed to the plant operators to not improve performance since it would hurt the community's chance to get a grant.

The influence of free federal money did not end once the grant was received. The existence of federal funding encouraged local governments to believe that the financing of wastewater treatment is not primarily a local responsibility. This theme was further impressed on plant operators even after they received notification of grant funding. The final approval for all aspects of design and construction rested with the EPA. Although the EPA informed them of their responsibility for design and construc-

tion, local communities were rarely held accountable when problems arose.

Numerous reports have documented the poor controls over federal grant money.[20-22] The U.S. Comptroller General reported that grant money had been used to finance the construction of such aesthetically valuable features as a red tile roof, stucco exterior, decorative arches, reflecting pool and a mosaic tile fountain — all at wastewater treatment plants.[20] The GAO found that "local agencies — the grantees — have not had adequate financial management systems to provide efficient and effective accountability and control over funds received from the agency."[21]

One town manager admitted that his town was "relatively poor and was not satisfied with merely constructing a plant whose design was compatible with existing surroundings; it wanted the facility to serve as a catalyst for upgrading the area."[20] This particular plant was surrounded by a 15-foot high red tile/stucco fence that cost $200,000. In another example, federal grants paid for 55 percent of the cost of a $30,000 mosaic tile fountain constructed for the sole purpose of displaying the effluent from an advanced wastewater treatment plant. The program enabled some local wastewater treatment agencies to expand their operations without having to spend local money or risk the political consequences of using local money.

The concern over "our own money" is not unusual. Initially, Congress and the EPA treated lack of funding as an acceptable excuse for noncompliance. Municipalities learned that if federal funds were not available, they would not be forced to comply with the deadlines of the FWPCA. Section 301(i)(l) of the CWA allowed plants not meeting the 1977 deadlines as a result of inadequate federal funding to request an extension until July 1, 1983. The act stated that if construction were required to meet discharge limitations, but "construction cannot be completed within the time frame . . . or the United States has failed to make financial assistance under this act available in time," the Administrator or the state could issue a permit containing a compliance schedule to have the plant construction completed "no later than July 1, 1983." Thus, the act clearly placed responsibility for funding with the federal government, and the failure of the government to provide funds was an acceptable excuse for noncompliance.

Local officials, who realized that a new wastewater treatment plant provided economic benefits to the community as a whole, were faced with a dilemma. Deliberate noncompliance, although technically illegal, was difficult to spot and rarely subject to enforcement, especially if a plant was waiting for a federal grant. In addition, money could be saved by reducing compliance efforts. Compliance, on the other hand, required local O&M expenditure and could actually hinder the chance of receiving a federal grant. As a result, the primary goal of operation usually centered on the ability of the local plant to receive outside grant funding and thereby minimize local costs.

Noncompliance moved a community farther up on the state priority list and encouraged state officials to provide construction funds to improve compliance through the construction of a new plant. Some officials believed that it was impossible to achieve compliance until a grant was received and, therefore, did not try to improve performance. Others refused to participate in the CCP study fearing that the study might improve plant performance and diminish their chances for a grant.

In 1978, the EPA issued its *Interim National Policy and Strategy for Construction Grants, NPDES Permits, and Enforcement Under the Clean Water Act*, which outlined necessary enforcement strategies to improve compliance.[8] One of the policy's goals was to encourage movement into the grant process by notifying POTWs that if they did not apply for a grant within a certain time period, they would lose the opportunity to receive a grant. In addition, MWTPs with a history of delay despite EPA pressure to comply, and MWTPs that had been determined to have adequate physical structures capable of meeting effluent limitations but were not meeting those limitations, were suggested as candidates for referral for judicial action.

The Construction Grants Amendments of 1981 ended reserve-capacity funding and reduced the federal share of design and construc-

tion costs to 55 percent. The 1984 National Municipal Policy warned communities that continued lack of federal funding would not be accepted as justification for noncompliance.

This policy marked a fundamental shift in the agency's compliance efforts. Federal grants were still intended to promote compliance, but a community's lack of funding was no longer an allowed excuse for noncompliance. Plants were warned to meet regulations without federal funding or face enforcement. But much damage resulted from the previous connection between compliance and funding. Plant owners and operators were constantly reminded of the connection. Congress and the EPA — through previous policy that included the massive outlay of funding, lax enforcement and repeated deadline delays — impressed local officials with three themes:

- The outlay of free money was an admission by Congress that it was the federal government's responsibility to construct wastewater treatment plants.
- Deadline delays indicated that local governments would not be penalized for the federal government's failure to provide enough money or the EPA's failure to provide enough guidance.
- Lax enforcement clearly demonstrated that compliance was not important enough for the EPA to justify enough time and manpower to seek out those plants that were not complying.

The established connection between funding and compliance hindered the goals of the CGP. Rather than encourage compliance through federal funding, it created perverse incentives that made it advantageous in the eyes of owners and operators *not* to comply or even attempt to improve performance.

Congress and the EPA intended the CGP to serve as a temporary form of assistance designed to encourage compliance, not as a perpetual source of funds for wastewater treatment plant construction. That the CGP was not intended to be a continuous source of money was clearly stated to all participants from the very beginning of the program. In 1987, then President Ronald Reagan attempted to block passage of the 1987 CWA amendments. His veto message, addressed to the House of Representatives, stated his reasons for objecting to continued CGP funding:[23]

> *The Clean Water Act construction grant program, which this legislation funds, is a classic example of how well-intentioned, short-term programs balloon into open-ended, long-term commitments costing billions of dollars more than anticipated or needed. Since 1972, the federal government has helped fund the construction of local sewage treatment facilities. This is a matter that historically and properly was the responsibility of state and local governments. The federal government's first spending in this area was intended to be a short-term effort to assist in financing the backlog of facilities needed at the time to meet the original Clean Water Act requirements.*

Operator Knowledge & Training. In conjunction with the CGP, the EPA worked closely with local officials and state agencies to ensure that communities had the technical expertise to operate their wastewater plants. The EPA-funded CCP study revealed that compliance was best promoted when regulators enforced performance standards and abstained from recommending particular compliance methods. When local officials followed regulatory technical advice, they believed that they were meeting performance requirements regardless of actual compliance to limitations.

However, specific recommendations provided by the EPA or state agencies were not always correct and did not always result in compliance or improved effluent quality. For example, staffing estimates provided in EPA publications sometimes varied from field usage, often exceeding what was necessary.[24] The resulting salary expenditures reduced the funds otherwise available for training and other O&M expenses. The CCP study consultants made the following observation:[25]

> *An aspect of regulatory agency activity that confused owners and redirected their activities away from achieving required effluent quality was that of providing specific operations or maintenance recommendations. Owners often relig-*

iously carried out these recommendations and were subsequently confused and frustrated when they found that they were facing still more requirements when compliance was not achieved. Additionally, assistance was perceived by the facility owners as a relief from local responsibility for effluent compliance.

The EPA's technical assistance and operator-training programs were designed to foster the development of a national base of skilled water pollution control personnel and technical information materials that would help protect the federal investment in wastewater treatment facilities.[26] The Water Quality Improvement Act of 1970 provided financial support for an EPA pilot-operator training program. Working with states, local communities, educational institutions and trade associations, the EPA provided a wide range of training programs that included management training, advanced treatment training, general skills workshops, informational seminars for local officials, correspondence study programs and preventive maintenance training.

Since the goal was to develop self-sufficient operator-training programs, the EPA recommended providing for on-site technical assistance that would be given by experienced O&M personnel, preferably state employees. The EPA discouraged states from using contract assistance approaches; instead it promoted an approach for self-sufficiency that could be acheived by states hiring qualified technical assistance personnel in state training centers or other responsible state program offices.

The CCP study determined that many local plant operators authorities did not realize the potential impact of operator actions on plant performance and, therefore, did not place an adequate emphasis on operator knowledge and training. Evident in the corrective recommendations issued by the study was the belief that operator education would receive more emphasis if its importance were recognized.

However, local participation in the CCP study was not mandatory. Local officials or operators who had reasons not to seek improvements at their plant would have reasons not to participate. The consultants noted that some municipalities refused to participate since they were reluctant to invest or focus their attention toward existing facilities while external construction grant funds were available. One official stated that "at this time it would be senseless to spend additional money on our present sewer treatment plant. Our plant is inadequate and has never functioned properly."[25] Even the possibility of future grant funding discouraged plant operators and local officials from improving operator training or plant performance.

The EPA's advice was not limited to the O&M of existing plants. Local officials in Garland, Texas, accepted an EPA recommendation and adopted an innovative technology. Under the CGP, certain innovative or alternative technologies received increased grants relative to conventional treatment methods. Garland took the extra funding and built a $30 million physical-chemical treatment plant. Seventeen days after startup the plant failed. William E. Dollar, Garland's Director of Public Works, claimed that "because this was such a new process, we depended on . . . the EPA to let us know" about the technology.[27] Initially, the EPA provided glowing recommendations about the technology. After the plant failed, the EPA sued Garland for failure to meet discharge permits. Garland sued the engineers involved with the project and again asked the EPA for help. Dollar claimed the town spent "almost a year in numerous meetings trying to get clear directions from the EPA." Eventually, Garland was forced to seek the help of an engineering consulting firm.

Like federal funds through the CGP, technical advice was provided by the EPA to foster compliance and local self-sufficient wastewater treatment programs. But, also like federal funding, federal technical assistance encouraged states and communities to view wastewater treatment as a federal responsibility. In an environment of free technical advice, lax enforcement and unclear assignments of responsibility, communities had little incentive to develop self-sufficient technical capabilities.

Design, Construction & Maintenance. The CCP study revealed that there were numerous design, construction and maintenance problems present in federally funded facilities. The ex-

tent of these design failures was particularly surprising given that plants with obvious design flaws were excluded from study. Design improvement and increased flexibility and controllability of equipment were the two physical plant recommendations that resulted from the CCP study.

Under the CGP, wastewater treatment plants were designed for a 20-year life. As a result, treatment requirements needed 20 years after the startup of the plant had to be estimated before design or construction began. Although the lack of accurate knowledge about future treatment needs would seem to have encouraged the use of flexible designs that would accommodate unpredicted changes, this did not appear to happen. As a result, plants ten years into operation may be restricted by equipment designed on the basis of an inaccurate future-needs prediction.

CGP regulations required EPA or state-agency approval of all design changes and construction upgrades of federally funded plants. The process of receiving design and construction approval from the EPA was a long and complicated process. Review of the three-step grant application could add as much as six years to the construction process; delays averaged two to four years.[28] In addition, some communities waited on state priority lists for ten years before receiving notification of grant eligibility. The enormous demand for grants, combined with lengthy regulatory reviews, hindered the EPA's ability to provide more frequent design reviews. In contrast, communities constructing plants without federal assistance (and therefore without EPA design restrictions and review delays) were free to employ more flexible designs, such as modular plants, that could accommodate future upgrades.

The EPA's 1982 *Handbook: Identification and Correction of Typical Design Deficiencies at Municipal Wastewater Treatment Facilities* identified many of the problems relating to restricted operation or lack of flexibility.[17] The handbook's intent was to provide "design engineers with guidance that will make their designs more operable and maintainable at less cost, as well as more flexible in providing adequate performance." The introduction to the handbook stated that design deficiencies contribute to a decrease in the flexibility of plant process control. Design flaws addressed in the handbook that relate to the lack of flexibility included:

- Inadequate process and operation flexibility;
- Inadequate consideration of seasonal impacts on operating efficiency;
- Inadequate estimation of present and future flows;
- Inability to adjust and control process equipment in response to changes in waste characteristics; and,
- Inadequate consideration of maintenance needs including: the lack of provision for by-passing flow, inability to dewater tanks for repair and the lack of a back-up unit.

The handbook also identified the problems resulting from adopting 20- to 30-year design lives. In many cases, the predicted design flow or treatment capability was based on inaccurate data. Numerous problems have resulted from errors of basing design on average flow and loadings (rather than peak), as well as from the failure to recognize the connection between treatment ability and seasonal influences when designing outdoor lagoons.

The 1980 GAO report stated that:[22]

> *Accountability under EPA's construction grants program is complicated by the many parties involved in the design and construction of a treatment plant: EPA regional officials, state regulatory agencies, municipal officials (the grantee), design engineering firms, industrial contributors, and finally, construction contractors and sub-contractors.*

The report concluded that clear assignments of accountability would improve performance without the need for additional federal funding.

At the time the GAO report was published, the CWA placed responsibility for proper plant performance on local officials. However, local governments were prohibited from implementing design plans that did not meet the approval of state and EPA officials. When plants failed to meet design expectations, local

officials blamed inadequate state and federal reviews. State and federal officials claimed that their reviews of design plans were never intended to thoroughly evaluate the design's ability to provide proper wastewater treatment.

The design and construction of wastewater treatment plants under the CGP was hindered by an inability to identify and hold accountable those responsible for providing inaccurate or improper design advice. As a result, construction and design became a lengthy and complicated process that frequently resulted in the construction of improperly designed plants that were incapable of producing effluent in compliance with NPDES effluent permits. The same problems existed in relation to providing technical training and O&M advice. As a result, plant operators could not guarantee the accuracy of advice furnished by regulatory agencies, local officials or private parties.

The inability to identify and hold accountable those responsible for providing inaccurate or improper advice was compounded by the availability of extensive federal subsidies for construction costs and training programs as well as the failure of regulatory agencies to enforce NPDES effluent permits. Since local governments were financially responsible for only a fraction of the construction and training assistance, they had little incentive to ensure the efficient expenditure of wastewater funds. In addition, since local governments were rarely held accountable for poor plant performance, they had little incentive to expend local resources to improve the accuracy of available advice or to seek new sources of information.

In sum, the failure of the CGP to encourage self-sufficient local government wastewater treatment programs resulted from two factors:

- The CGP process was marked by an inability to identify and hold accountable those responsible for the provision of inaccurate or improper design, construction or operation advice.
- The CGP failed to provide incentives for local communities to pursue efficient wastewater treatment plant construction and operation.

Over time, recognition of these problems in an environment of decreasing federal funding and increasing treatment needs led Congress, the EPA and local governments to explore new ways of providing municipal wastewater treatment.

Changes in State & Federal Approaches to Municipal Wastewater Treatment

State Revolving Funds. Title VI of the 1987 Water Quality Act introduced the State Revolving Fund Capitalization Grant Program (SRFCGP) that was designed to gradually convert wastewater treatment plant funding from federal grants to state loans. While the act extended the grant program by authorizing EPA allocation of construction grants until 1990, it also set forth a schedule for ending all forms of direct federal funding by the end of fiscal 1994. By that time, states were expected to have self-sufficient wastewater treatment programs. However, the federal government has retreated from this intent.

Congress recognized that the transition to state responsibility would be difficult since states were accustomed to decades of federal support (some of them prior federal attempts to develop self-sufficient state programs). Therefore, Congress encouraged states to develop self-sufficient funding programs by authorizing federal capitalization grants for approved SRFs.[30] A revolving fund provides a continuous source of funds if the fund's proceeds are loaned, not granted, to communities. The funds are sustained through repayment of loans made from the capitalization grants.

Congress allocated $8.4 billion for capitalization grants between 1990 and 1994. States were also allowed to capitalize their SRFs with a portion of their construction grant funds that were available through 1990. In order to qualify for capitalization grants, states were required to provide a 20 percent match to the federal share and authorized to reserve four percent of the grant funds for administrative costs. In addition, states had to establish an EPA-approved state environmental review program (SERP) and all SRF-funded projects had to undergo SERP reviews.

To establish a SRF, a state had to submit an application form that included a description of the SRF structure and an Intended Use Plan

(IUP) that specified how the state will use grant funds to help local governments meet the requirements of the CWA. Each year thereafter, the state must submit an updated IUP and an annual report. These reports detailed the state's efforts to comply with SRF administrative requirements (such as federal accounting and auditing procedures) and ensured that projects funded by SRF funds have complied with at least 46 different federal regulations including 16 different Title II requirements that were carried over from the CGP that specifically address wastewater treatment, as well as at least 30 different federal laws and executive orders attached to federal funding.[30] Title II requirements applied only to projects funded by the federal capitalization grant, not to projects funded by state matching funds or funds resulting from repayment of earlier SRF loans.

Unlike the CGP, states were primarily responsible for ensuring that communities adhere to funding regulations. Rather than directly review individual projects, the EPA would oversee compliance with funding regulations through its review of state annual reports. If the EPA's Office of Inspector General determined that a state had not administered the SRF in compliance with federal standards, federal capitalization grants to the state's SRF would be suspended. If the state fails to take corrective action, the capitalization grants would be awarded to another state.

Funds from a qualifying SRF must be loaned to communities at or below market interest rates. Negative interest and grants were prohibited. While SRFs could be used to fund pollution control projects other than wastewater treatment, states must ensure that funds were first directed toward facilities not yet in compliance with the CWA. After complying with this first-use requirement, states could assist the development of other water pollution control projects such as the prevention of nonpoint source or estuary pollution. All states have met first-use requirements.

Since the SRFCGP converted federal grants to loans that had to be repaid, local officials were encouraged to provide the best treatment at the least cost and to avoid unnecessary features. The SRFCGP also increased the flexibility of wastewater treatment funding by allowing SRFs to be used to provide guarantees for local loans, to purchase municipal bond insurance, or to refinance existing local government wastewater treatment debt obligations. In addition, by providing a renewable source of wastewater treatment funding, Congress attempted to end continued state and local government reliance on federal funds and finally establish self-sufficient treatment programs.

Detailed information on the ability of SRFs to finance efficient wastewater treatment is not yet available.[31] However, initial findings indicate that the need to repay loans is resulting in the construction of more appropriate and efficient wastewater treatment facilities. For example, some communities that were encouraged by the CGP to finance expensive, centralized facilities are now turning to low-cost on-site systems.[12] State officials expect SRF financing to result in the construction of lower-cost facilities.

Although the SRF appears to be an improvement over the CGP, numerous aspects of the SRF still inhibit efficient investment in MWTPs capable of meeting effluent standards. States claim current program regulations hinder the ability of communities to finance appropriate wastewater treatment methods. SRF loans can only be used to purchase land if the land itself is part of the treatment process. Thus, communities may receive assistance for the acquisition of land to develop wetland filtration projects but not for treatment plants or easements for sewer collection systems. The EPA included land purchase restrictions in the CGP to prevent the purchase of unnecessary land that could later be used for other purposes. Forty-two states responding to a 1992 GAO SRF survey reported that land restrictions should be waived.[12] According to an official in charge of Florida's program, the cost of land for unsewered communities can represent about 20 percent of the project's cost. The GAO noted that states were capable of determining necessary land requirements and concluded "that an across-the-board restriction on the eligibility of land purchases for SRF assistance is counterproductive for many local governments."[12]

The GAO also criticized SRFCGP restrictions that limited loan terms to 20 years and the EPA's failure to provide knowledgeable EPA

staff to assist states in developing SRFs. Currently, communities receive 20-year loan terms regardless of the design life of the facility.

As a result, some communities may find it difficult to finance facilities with longer design lives while others may find that loan repayments extending beyond the design life hinder the ability to upgrade a previously financed plant. The GAO recommended matching loan terms to the estimated design life of the facility. The GAO also concluded that states lack the expertise to manage SRFs and suggested that the EPA increase state technical and administrative assistance through additional training and hiring of EPA regional staff. Such recommendations were likely to discourage states from accepting responsibility for SRF administration.

The SRFCGP also restricted a community's ability to provide the most appropriate treatment method for its needs. For example, SRF loans could not be used to finance the portion of a wastewater treatment facility owned by the private sector. In addition, Congress continued to restrict the flexibility of the treatment selection process through the carry-over of numerous CGP Title II provisions that included requirements that local governments select treatment technologies meeting EPA approval. Communities also had to demonstrate that they considered innovative or alternative treatment methods for projects funded from funds equivalent to those granted by the federal capitalization funds.

The disadvantages of the SRFCGP resulted from the ways in which it resembled the CGP and discouraged local governments from developing self-sufficient wastewater treatment programs. This effect resulted from the continued federal regulation of the wastewater funding process administered by the states and of the treatment selection process undertaken by local governments. According to the WEF Executive Committee, "administrative burdens imposed by the Federal government upon obtaining revolving loan funds . . . add unnecessarily to facility costs, discourage use of these funds by smaller municipalities, delay the construction of facilities, and reduce states' flexibility to allocate loan funds according to their own priorities."[32]

Public-Private Partnerships. As a result of the cumbersome requirements of federal funding, decreased funding, stricter wastewater regulations, and decaying infrastructure, many local and state governments have turned to various forms of privatization or public-private partnerships to solve their wastewater treatment problems. Studies investigating capital-intensive services, such as wastewater treatment, have revealed that economics (either the lack of eligibility for a federal grant or the high cost of running a plan) is commonly cited as the number one reason for considering private-sector involvement.[33] However, communities that have increased the private sector's role in either the production or provision of wastewater treatment have found that economics is not the only factor to consider. The EPA lists five basic reasons for communities to consider public-private partnerships:[4]

- Access to more sophisticated technology;
- Cost-effective design, construction and/or operation;
- Flexible financing, including the use of private capital;
- Delegation of responsibility and risk; and,
- Guaranteed cost.

Even during the height of the construction grants program, some communities elected to forgo federal funding and use their own funds to meet their treatment needs. EPA Assistant Administrator of Water Lawrence Jensen described the effects of the CGP program:[34]

Federal dominance inadvertently fostered an atmosphere of passive dependency. Action on existing needs and planning for future growth was thwarted as some municipalities stood in the federal waiting line. Conversely, many communities were not interested in this waiting game and proceeded to find suitable financing solutions on their own. These communities overcame obstacles and realized substantial benefits as they proceeded, thus making the grants trade-off worthwhile. Some notable advantages include cheaper and more efficient construction, ability to select local design preferences, greater responsiveness to economic growth, fewer procedural requirements, enhanced flexibility to address fu-

TABLE 3
The Wastewater Treatment Plant Ownership/Operation Continuum

	Traditional Treatment	Contract Services	Turnkey Facility	Developer Financing	Privatization	Merchant Facility
Decision to Provide	Public	Public	Public	Public	Public	Private
Design	Public	Public	Private	Either	Private	Private
Financing	Public	Public	Public	Private	Private	Private
Construction	Public	Public	Private	Either	Private	Private
Ownership	Public	Public	Public	Either	Private	Private
Operation & Maintenance	Public	Private	Either	Either	Private	Private

Note: Based on a table from Ref. 6.

ture changes, and greater certainty as to the timing of services to customers.

A 1989 study of wastewater treatment privatization conducted by Laurence J. O'Toole documented some of the beneficial effects of greater private-sector participation:[35]

> *Privatized design and construction of wastewater treatment facilities proceeded more smoothly, as perceived by participants, and much more quickly (by more than two years, on average) than in the grant-funded setting. Furthermore . . . output measures of effluent quality and of compliance with regulatory standards showed that privatized facilities do not suffer by comparison with . . . counterparts.*

Private involvement in providing wastewater treatment can take several different forms. Table 3 displays the most common variations of public-private partnerships. Represented are six aspects of wastewater treatment that can be the responsibility of either the public or private sector. The six aspects together encompass the entire process of wastewater treatment. The public-private continuum displays the strictly public involvement of traditional treatment on the left side, and the entirely private involvement of a merchant facility on the right. At least four combinations of public-private involvement make up the continuum between these two extremes. Not all partnerships follow these strict assignments of public or private responsibility. The wide variety of approaches allows public-private partnerships to be tailored to the specific needs of the municipality and the capability of the private sector.

Municipal wastewater treatment facilities have been constructed under each of the six different combinations of public-private partnerships displayed. The most common choices, excluding the traditional public-only model, are contract services and turnkey facilities. Privatization and merchant facilities are much less common after the Tax Reform Act of 1986, which reduced the tax incentives for private ownership. Frequently, developer financing results from a community's need to accommodate growth. Merchant facilities are characterized by full private-sector control of all service aspects, including the initial decision to provide. A merchant facility providing wastewater treatment is not common but may be more prevalent in the future as private firms offer to provide treatment plants to small unsewered communities or to communities that cannot afford to upgrade or replace their current plants.

Contract Services. The most common form of wastewater privatization is a contract under which a private firm takes responsibility for the day-to-day O&M of a publicly owned plant. The O&M contract signed by the city of Leominster, Massachusetts, illustrates several advantages of contracting out. As with all

forms of private involvement, this contract was unique, but most service contracts contain similar key features.

Leominster, with a population of 35,000 people, completed construction of a new plant in 1983. The $23-million plant was financed through the CGP, with the city contributing less than $3 million of the construction costs.[36] Leominster sought an O&M contract for the new plant for two reasons. The new plant employed advanced wastewater treatment (AWT) technology and its O&M would require more money and highly trained personnel than were available. The new sophisticated technology of the AWT plant was needed to remove phosphorus and ammonia, but it was also expected to require $1.2 million in O&M costs every year — a substantial increase over the previous $450,000 per year required for the older plant.

Leominster contracted with an environmental services firm to ensure that the plant was adequately operated and maintained for less than $800,000 per year (for a cost savings of about 30 percent). The contractor offered jobs to all seven of the plant's existing employees and provided a manager, who was selected in part by the town. As part of the operating agreement, the contractor provided a performance guarantee and liability insurance. For the initial six months of the contract, the firm provided 15 specialists to evaluate and improve not only O&M, but also nontechnical areas such as employee and community relations, administration and accounting.

The biggest change for employees was the installation of an automatic dialing alarm system, which the contractor had used at several other plants. The system automatically dials a technical assistance hotline when a treatment malfunction occurs. An electronic voice informs the operator of the location and type of problem. Leominster discovered that the automatic alarm system was more reliable than a single manned operation. The plant met effluent quality requirements within three days of startup. Technical expertise provided by the contractor saved Leominster an estimated $400,000 per year.

Contract operations illustrate several advantages of private-sector involvement. Many communities like Leominster are unable to provide the technological expertise necessary to operate a treatment plant. In some cases, political or financial pressures prevent a community from hiring the necessary personnel since the salary of a qualified operator may exceed that of the local mayor. Using a private contractor to provide O&M, however, can be attractive even to those communities that are able to afford the expense of an operator.

Unlike an individual community, private contract firms are able to take advantage of economies of scale — cost savings from large-scale production. Thus, the contractor could lower Leominster's O&M costs by providing a centralized alarm system, the cost of which is shared by the firm's other clients. Such economies of scale also extend to the bulk purchase of treatment chemicals and emergency pumps or other specialized equipment as well as to personnel that can be transported to a site as needed.

Municipal operation of wastewater treatment is also hindered by unclear assignments of responsibility and accountability. In the public management arena, political power struggles have resulted in continued noncompliance while various levels and departments of local government argue over the relative importance of wastewater treatment and the best methods of ensuring proper operation. Under a service contract, a private firm usually takes responsibility for compliance with NPDES discharge permits. The contract can ensure that the private sector is held accountable for noncompliance by requiring payment of EPA enforcement actions. In addition, the daily responsibilities for payroll, insurance, maintenance and regulatory reporting are transferred to the private firm and accountability for these issues is guaranteed through the contract. The private firm has experience at addressing these issues as they relate to wastewater treatment and, unlike municipal operators, it is free to make adjustments in daily operation without waiting to first obtain permission for the changes from the local government.

Accountability provided in the contract ensures that a private operator will either meet the contract standards established by the local government or face penalties such as fines or contract termination. Furthermore, a private

firm has an incentive not only to meet the standards established by the contract, but to exceed them. If a private firm fails to operate and maintain a plant in the most efficient, low-cost manner, it may lose its contract to a firm that can. Unlike municipal operation, contract service is not a monopoly. Competition from other contract firms serves as a constant incentive to improve performance.

Turnkey Facilities. The turnkey approach takes private involvement two steps beyond contracting out and places design and construction responsibilities in the hands of the private sector. As noted in the CCP study, even the best O&M services cannot overcome poor design or construction defects. Placing responsibility for design and construction with the same party responsible for O&M ensures a coordinated approach. Under the CGP, it was difficult to hold anyone responsible for poor design or construction defects. In addition, since few incentives for proper O&M existed and poor performance was rarely punished, the extent of design and construction defects frequently remained hidden.

The CCP study, which focused on O&M improvement techniques, deliberately excluded plants with obvious design or construction defects. Nonetheless, the study found that latent design defects "were severely limiting the operator's ability to achieve maximum performance from the facility."[17] Under a turnkey contract, a private firm designs, builds and operates a wastewater treatment plant, but the ownership remains public. The coordination of design, construction and O&M forces the private contractor to consider the relationship between the individual steps. Accountability provided under contract and competition from other firms ensures that the contractor will not make sacrifices in one area if they increase costs or place an unacceptable burden on another.

The city of Casa Grande, Arizona, chose a turnkey approach to upgrade its wastewater treatment system for the sole purpose of providing higher-quality effluent for irrigation. The city relied on an aerated lagoon and stabilization ponds for wastewater treatment. Its effluent was meeting standards, and the lagoon had enough reserve capacity to serve the area for many additional years. However, the city had only a nine-hole golf course and wanted an 18-hole course. Lack of state-approved irrigation-quality water prevented expansion.

Two consulting firms worked together to design, construct and operate the new facility. Unlike Leominster, one of the contractors wrote specific O&M manuals and trained city employees to operate the new system but did not take over the daily O&M responsibilities. The new facility was not significantly more difficult for city employees to operate, and the firm's only continuing responsibility for O&M was to furnish on-call laboratory analysis and operational support when needed.[37]

Casa Grande's turnkey contract required that the plant's upgrade be compatible with future plant expansions. One of the consulting firms constructed an upgrade that could be operated in parallel with future upgrades and provided extra capacity in two of the capital-intensive areas (the chlorine and chemical buildings). As a result, Casa Grande obtained a wastewater treatment system that could change with the city's needs. The use of such modular designs may reduce construction costs by as much as 25 percent.[38] By contrast, CGP-funded plants were frequently inflexible and, therefore, difficult to adapt to communities' changing needs because of grant funding that encouraged over-design and over-construction.

A significant advantage of turnkey contracts is the ability of design and construction engineers to coordinate the design, material procurement and construction phases. Such coordination provides more flexibility in the construction process and allows designers and construction engineers to devote more attention to the possible use of innovative design or construction techniques.[39] Under the CGP, design was primarily controlled by design engineers with little input from the contractor. Coordination of the process, achieved by the turnkey approach, can minimize change orders and reduce friction. For Casa Grande, such coordination provided significant savings:[37]

The turnkey approach saved eight weeks in the bidding/award/mobilization process alone. Additional savings to the tight construction schedule

were realized when the equipment procurement process began at the 50 percent design level . . . The turnkey approach also eliminated delays during construction by putting the engineering and construction staff in a single trailer at the site, thereby trimming communications time and reducing delays due to submittal preparation, transmittal and review.

Developer Financing. Developer financing places another aspect of wastewater treatment plant construction in the hands of the private sector, although developer responsibility for finance is not always accompanied by developer responsibility for design, construction or maintenance. In its most common forms, developer financing is characterized by private-sector design and construction of a wastewater facility that is turned over to the local government when completed, or as developer assistance that is provided directly to the local government for the construction of wastewater treatment facilities.

Developer financing is a way of "requiring growth to pay its own way."[40] Given that new communities generally do not have a tax base to finance the construction of public facilities such as wastewater treatment plants and schools, developers are encouraged to participate in financing public projects. In some areas (such as Escondido, California), financial contributions to the construction of a wastewater facility have been required in exchange for the permission to develop or the right to reserve a portion of future sewer capacity.

The city of Escondido faced three problems. The current wastewater treatment plant was operating at capacity and the city was growing. Furthermore, the city faced enforcement from the state for poor plant operation and a lawsuit from a neighboring town for failure to fulfill a sewer service contract.[6] Escondido needed an upgraded plant, increased sewer capacity and relief from the enforcement efforts and lawsuit.

As a result of several voter referenda, Escondido was prohibited from using bond financing, user fees or other forms of financing to upgrade its plant. By selling the rights to future sewer capacity to developers and citizens before the capacity was available, Escondido financed the entire cost of the plant upgrade and sewer capacity expansion and actually had funds remaining for future needs.

In return for the needed access rights to the sewers, developers and individual citizens paid for contracts with the city that designated the amount of capacity purchased. Initially, access rights were sold for $1,650 per equivalent dwelling unit (EDU). One EDU is the equivalent of 270 gallons per day, which is the estimated daily amount required for a family of four. Seven years later the price of one EDU had risen to $3,300.

In this arrangement, the city of Escondido was responsible for individual contracts and required all people seeking access to the sewers to purchase a contract with the city. The first purchasers who bought rights before the plant was built received an exemption from future increases of connection fees as an incentive to help finance the plant. Escondido also signed a contract with the neighboring city of San Diego to provide additional sewage treatment for that city. Private-sector involvement was limited to the purchase of access rights that financed the entire project.

Full Privatization. While all forms of public-private partnerships can be described as forms of privatization, full privatization has its own unique meaning. When a facility is described as being fully privatized, the responsibility for the design, construction, financing, ownership and O&M rests entirely with the private sector. The popularity of full privatization decreased due to the changes in tax laws in the mid 1980s. These changes made private ownership much less attractive and hindered the ability of communities to issue tax-exempt bonds to finance privatized facilities.

The 1986 Tax Reform Act eliminated or reduced several incentives for private investment in wastewater treatment facilities. Previously, private investors received a ten percent Investment Tax Credit for the purchase of certain types of depreciable property. For a typical wastewater treatment plant, this credit applied to 80 to 90 percent of the total cost.[41] The 1986 act also lengthened depreciation schedules. Depreciation allows an investor to recognize the declining value of an asset as it ages by writing off a percentage of the asset's cost each year of the asset's life. The 1981 Accelerated Cost Re-

covery System encouraged investment by allowing investors to depreciate an asset over a time period shorter than its useful life. For wastewater treatment plants, 80 to 90 percent of assets qualified for five-year depreciation schedules. By lengthening the depreciation period of treatment plants from five to 15 years, the 1986 act reduced the amount of cash available to support an investment in its early years.

Tax reform also hindered the ability of local governments to issue tax-exempt bonds to finance wastewater treatment. Until 1968, all income received by investors from state or local bonds was tax-exempt. The Revenue and Tax Expenditure Control Act of 1968 eliminated the exemption for most bonds if more than 25 percent of the bond proceeds were used by a nongovernmental entity in a trade or business and bond repayment for more than 25 percent of the bonds was guaranteed by property taxes or revenues generated by the business. Bonds used to finance projects failing either test were tax-exempt. Bonds passing both tests were defined as taxable industrial development bonds (IDBs). Exemptions from these two tests, known as the *use of proceeds* and *security interest* tests, were granted for numerous projects, including wastewater treatment.

The 1986 Tax Reform Act redefined IDBs as public activity bonds (PABs) and reduced the percentages of the use of proceeds and security interest tests to ten percent. For example, even a *publicly* owned wastewater treatment plant serving industrial users under a contract that differed from the contract provided to the general public could end up in the PAB category if the industrial use exceeded ten percent of the plant's capacity (average industrial use is 16 percent).[42] Wastewater plants operated under contract operations by the private sector were also classified as PAB projects unless the contract life was limited to no more than five years, the private operator's compensation was not based on profit sharing and the local government had the option of terminating the contract at the end of three years without penalty.

Although the 1986 act retained the previous tax exemption for wastewater treatment plants, such projects had to comply with state volume caps that limited the amount of PABs that a state could issue to the equivalent of $50 per resident, with a minimum of $150 million allocated to each state. Unlike airports and docks, which are excluded from volume caps, wastewater treatment facilities must compete with other projects such as multifamily housing and student loans for the limited amount of tax-exempt financing available under the cap.

A report prepared for the EPA's Office of Wastewater Enforcement and Compliance contended that restrictions on private wastewater treatment contracts by the 1986 Tax Reform Act might "prevent the parties involved from negotiating a contract which is more cost effective for the local government."[42] As a result of these tax law changes, private involvement since 1986 more often takes the form of a turnkey agreement with a private O&M contract.

Several proposals have been made to modify or reclassify bond legislation to provide communities with greater flexibility. The Anthony Commission — an ad hoc Congressional committee established by Representative Beryl Anthony in 1988 to study the effects of the 1986 Tax Reform Act on the municipal bond market — recommended the creation of tax-exempt public-activity bonds that could be used to finance privately owned and operated wastewater plants. The commission's proposals extended the allowable operation contract length from five years to the facility's economically useful life, but prevented private owners or operators from taking any cost recovery deductions.[43] In 1991, Senator Pete Domenici introduced federal legislation to create infrastructure bonds for environmental infrastructure projects that would exempt such projects from volume caps and allow accelerated depreciation. The EPA's Environmental Financial Advisory Board recommended a similar approach for environmental facilities mandated by federal law.

In November 1993, legislation to remove an impediment to private investment in municipal wastewater treatment plants was introduced in Congress by Senator Frank Lautenberg (S. 1681) to amend the FWPCA to permit some privately owned public treatment plants to be treated as publicly owned treatment plants. Because privately owned treatment works generally must meet more onerous treatment requirements (by providing uniform

regulatory treatment), the act could help attract private investment to municipal facilities.

Two privatized wastewater treatment plants built before the tax-law change are located in Auburn, Alabama. Auburn was low on the state list to receive funding through the CGP even though both of the city's wastewater treatment plants were operating at capacity and were having trouble maintaining effluent quality.[6] Auburn is typical of many towns in that it waited years to receive federal money, only to find that it had to build plants without federal assistance. After 12 years of waiting for the federal grant, Auburn finally decided to investigate the privatization option to finance two new plants with specially issued bonds. An engineering and consulting firm was hired to design, construct, operate and own the new plants. Although the firm was not solely responsible for the financing, it did contribute $10 million in equity to reduce outstanding debt and debt-service payments. In return for its involvement, the consultant received the benefits of the favorable tax laws and the city's previous wastewater employees were all trained and offered positions in the new plants. The city expects to save $25 million over the life of the 25-year contract.

In 1993, the EPA selected wastewater treatment plants in three cities (Franklin, Ohio; Indianapolis, Indiana; and, Silverton, Oregon) as pilot projects to test privatization under Executive Order 12803. The April 1992 order established federal policy in favor of requests by state and municipal governments to sell or lease infrastructure enterprises that had received federal grants, explicitly including wastewater treatment plants. By allowing local governments to reimburse the federal treasury for only the undepreciated portion of federal grant financing (and not the entire amount), the order removed an important impediment to privatization.

A private firm has offered to buy one of the pilot projects, the Franklin (Ohio) Area Wastewater Treatment Plant. Under a 20-year service agreement, the firm would maintain constant inflation-adjusted rates and charge a negotiated fee to the municipality, which would collect customer fees. The original cost less depreciation value for the plant is $6.8 million, which is close to the transfer price of $6.1 million that the contractor will have to pay to service the four-million-gallon-per-day plant's debt and cap the rates at their present levels. The transfer is expected to be completed by mid-summer 1994.

Privatization & Performance. The various benefits of privatization result from the clear identification of responsibility and accountability, which accompanies private-sector involvement, and from incentives naturally arising in a competitive environment that encourage efficient construction and operation. *Clear assignments of responsibility are linked to accountability through a privatization contract.* In an extensive study of capital-intensive privatization, John G. Heilman and Gerald W. Johnson concluded that privatization shifts "[a]ccountability mechanisms . . . from traditional local political processes to the contract and to the ongoing partnership between the privatizer and the authority."[44]

This transfer from the political process to the contractual setting places the provision of wastewater treatment in a competitive environment where private firms continuously compete to be the best provider of a community's needs. It also removes the day-to-day responsibility for protecting the community's wastewater needs from public officials and places it with the private sector, where it is protected by a partnership contract and legal enforcement.

Under a proper privatization contract for design, construction and O&M, the responsibilities of both parties are clearly defined. The municipality knows what it can expect from the private firm and the private firm knows what it must do to fulfill the contract. In addition to responding to the threat of legal action such as contract termination or financial penalties for poor performance, the private sector must also respond to financial constraints. Not only does the private firm realize that lack of funding will not be an acceptable excuse for noncompliance, but the private firm has an incentive to perform the service at the least possible cost while still meeting the terms of the contract. If a firm fails to operate an efficient plant, it may lose its business to one that does.

Clear assignments of responsibility, combined with proper accountability, can ensure

that incentives exist for proper performance. In the federal-funding environment, incentives exist that encourage noncompliance. These incentives encouraged the pursuit of lower local O&M expenditures, federal grant funds and new MWTPs for the purpose of improving local economic conditions. Since local governments were not held accountable for improperly designed plants, there were few incentives to seek proper performance and compliance with NPDES effluent permits. If it is not possible to identify and hold accountable those responsible for poor compliance, such incentives will remain a tempting course of action for local governments.

Privatization also provides incentives that encourage the constant pursuit of improved wastewater treatment technology. Economist John Donahue has noted that the benefits of innovation are not only related to improved responsibility and accountability for risks and profits:[45]

> *For a municipal agency, the potential* payoff *for innovation is limited to whatever lower costs or higher quality can be achieved within the city limits. Except in the biggest cities, it seldom makes sense for public works departments to make large investments in innovation. A private contractor, however, can claim proprietary rights to innovations, diffuse new methods through its operations, and use technological advances as a competitive edge to expand its market.*

Privatization provides a strong incentive for firms to develop improved treatment technologies and operation methods because the firm that does not constantly struggle to improve will be forced out of the market by others pursing improvements for their own benefit. Thus, privatization captures the benefits produced by competition and provides these benefits to the local government that remains protected from the risks of research and development by the privatization contract.

The avoidance of costly regulations and restrictions that accompany private involvement in providing wastewater treatment fosters another benefit. Removing many of the federal restrictions governing wastewater treatment construction places the municipality and the private sector into a partnership where both sides seek similar goals. Heilman and Johnson note that involvement of the EPA "introduces the at least partially competing values of the regulatory process."[44]

For example, competing values may encourage the EPA to compensate a community and its design engineers for regulatory burdens by approving expensive treatment plant designs. In addition, by fostering incentives that made it desirable not to comply with clean water regulations, the CGP discouraged local governments from improving wastewater treatment capacity and performance.

In the area of design and construction, the differences between competing goals and complementary goals is clear. O'Toole studied two plants in the same town — one CGP grant-funded and one privatized. He found that in a public setting "designers tend to view change order requests as a challenge to the quality of the original design and thus seek to defend their turf."[46] In comparing the contracting costs of two plants, O'Toole notes that "substantial transaction costs were accrued in negotiations with the grant-funded designer. However, in the privatized case, the builder and the designer had complementary incentives and early agreement was reached."

Privatization also avoids the necessity of regulatory and municipal reviews of design changes. Such reviews are conducted to ensure that changes meet federal quality and funding eligibility requirements. In a grant-funded setting these reviews can take weeks or even months. Privatization avoids design change delays while shifting responsibility for quality assurance and cost control from the regulatory agency to contractual guarantees.

If the construction and the financing aspects of the partnership are not connected to federal funds, federal rules and regulations attached to grant funds can be avoided. When money was in grant form it was easier to comply with such standards since someone else was paying for the majority share. But now myriad regulations including bidding procedures and pay scales significantly raise the cost of securing funding.

Restrictions that accompany federal funding include the Davis-Bacon Wage Act, which re-

quires that all workers on federally funded projects receive the prevailing wage rates of the area. O'Toole notes that this policy reduces wage competition and drives up the cost of construction. As a result of protests by construction firms, the Department of Labor reduced prevailing wage rates in 1983. For one treatment plant bid alone, the estimated savings from reducing the wage rates by an average of 35 percent was $300,000 to $500,000 or almost 13 percent of total job costs.[47]

Summary

Clearly, federal policy has had an impact on the effectiveness of municipal wastewater treatment. If the goal of these policies has been clean water, these policies have not always attained that goal (no matter their intent). For federal policy to be truly effective, it would need to integrate the following:

- Environmental and fiscal policy must be coordinated so that activities engaged in one policy arena would not be negated by the other.
- Goals and pollutant limits should be clearly stated, but the means to achieve them left up to states and communities.
- Fiscal policies should not narrow the range of solutions that should be available to communities — and allow for innovative solutions.

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Regulated Structural Peer Review

With the trend toward more complex structures and building codes, a properly conducted review of a structure's design can be of great help in avoiding disaster.

GLENN R. BELL & CONRAD P. ROBERGE

On June 19, 1992, mandatory requirements for independent structural engineering review of the design of new structures built in the Commonwealth of Massachusetts became part of the Massachusetts State Building Code (MSBC). The process, known as *peer review*, applies to the design of all structures over certain threshold limits of size or occupancy. The purpose of the review is to enhance public safety through independent verification that a new structural design appears to be conceptually correct and free from major errors. Under the MSBC provisions, the prospective "owner" of a new building retains a reviewing structural engineer, who is independent of the design engineer of record and others on project team, to conduct an overview of the structural design by checking the building's overall design criteria and the concept of the structural system, and by checking the design of a representative fraction of the structural elements. The review is not intended to be an exhaustive check.

The Commonwealth of Massachusetts is progressive in adopting such requirements for peer review — only one other state, Connecticut, is known to have similar requirements. But the concept of structural peer review is not new. It has been developed and promoted for over ten years by industry organizations concerned with mitigating the occurrence of life-threatening structural failures. Nationally, support for the adoption of structural peer review is growing rapidly, and other countries have had similar requirements for decades.

Two years after adopting MSBC requirements for structural peer review, many building owners and others (including some structural engineers) still challenge the need for, or the intent of, these requirements. Individual cases have demonstrated that there are many professional and procedural issues to face. And, while meeting the letter of the requirements, a peer review can be ineffective if it is not properly conducted according to the intent of the provisions. Peer review is now a logical part of the design/construction process made necessary by the evolution of the construction industry.

Historical Development

Building Failures & Structural Design in the United States. The current process of the struc-

tural design of buildings has evolved over the last century. In times past, building design involved:

- Simple methods and codes;
- Large factors of safety;
- Straightforward communication among all participants; and,
- Clear lines of responsibility.

Today, building design has become a complex business of producing high-performance structures involving:

- Complicated building codes and design specifications;
- Low factors of safety;
- Design/construction teams with many parties and unclear lines of responsibility; and,
- Often extreme financial and time pressures.

While the incidence of catastrophic, life-threatening failures has been relatively low, there were a rash of such failures from the late 1970s through the late 1980s, suggesting that something had gone awry. Among the major failures are the following:

Hartford Civic Center. In January 1978 the roof of the Hartford Civic Center Coliseum collapsed. Fortunately, no one was in the building at the time of the failure, but just a few hours earlier 6,000 persons had been assembled in the building for a sports event. An investigation revealed that a design error involving improper bracing of top chord compression members was the likely cause of the roof collapse.[1]

Willow Island Cooling Tower. In April 1978 a large section of a reinforced concrete cooling tower at Willow Island, West Virginia, collapsed while under construction, killing 51 construction workers. The collapse occurred because construction loads from a formwork system were imposed on the structure before the concrete had gained sufficient strength to support the loads.[2]

Kemper Arena. In June 1979 the roof of the Kemper Arena in Kansas City collapsed. Like the Hartford Civic Center, the Kemper Arena was densely occupied just before the failure but was essentially empty at the time of the failure. A high-strength bolted connection that linked the roof's space frames was subjected to a wind-induced rocking motion that was not accounted for in the design. This motion fatigued and loosened the bolts, causing them to fail under an intense wind and rain storm.[1]

Harbour Cay Condominium. The Harbour Cay Condominium in Cocoa Beach, Florida, collapsed in March 1981 while under construction, killing 11 construction workers. The building had been designed by two retired NASA engineers. One of them, who had performed most of the building's design, had little experience with building design. The other allowed his professional engineering seal to be affixed to the plans even though he was not familiar with the building's design.[3]

Hyatt Regency Walkways. In July 1981 two suspended walkways at the Hyatt Regency Hotel in Kansas City, Missouri, collapsed, killing 114 people and injuring hundreds of others. Investigations after the failure revealed that a critical hanger-rod-to-walkway-beam connection was never structurally designed. Responsibility for the connection was lost in communication between the structural engineer and the fabricator's detailer. Compounding this error was a construction-phase change in the walkways' hanger rod arrangement that essentially doubled the load borne by the critical connections.[1]

L'Ambiance Plaza. In April 1987 the partially completed 16-story L'Ambiance Plaza Apartment building in Bridgeport, Connecticut, collapsed while under construction. Twenty-eight construction workers were killed. While the cause of the failure was never conclusively determined, investigations by many organizations revealed several gross deficiencies in the structural design and the construction of the building. These problems were the result of a serious breakdown in communication, the unclear assignment of design responsibility, lack of quality assurance programs and other safeguards to public safety.[4]

While there are a myriad design and construction quality issues in all of these failures, a common quality step lacking in all of them is independent verification of the adequacy of the structural design. In typical United States practice, the assurance of the quality of a structural design rests almost entirely with the engineer of record. There are no regulatory checks as there are in other industries, such as airlines or pharmaceuticals. Many years ago, plan reviews by local building departments often provided thorough reviews of building structural designs and, in some cities like Los Angeles, building departments still provide comprehensive reviews. Generally, however, municipal building departments have neither the staff nor the funds to provide meaningful reviews of today's complex structural designs.

One characteristic of catastrophic building failures is that they tend to be low-frequency, high-consequence events. They are disasters that are infrequent, and while they receive great attention when they do occur, they are forgotten by many shortly thereafter so that high priority on their avoidance is lost. Consequently, building owners and the general public frequently do not value the importance of sound structural engineering.

In an industry where financial competition and time pressures are severe, responsibility is not well defined and communications may be unclear, this is a recipe for disaster. The structural engineering profession realizes that it has an obligation to take a stronger role in assuring public safety. That obligation is better served by exercising some form of self-regulation rather than by having such regulation imposed on the profession by others. Peer review can help mitigate these disasters.

Early Resolutions of Industry Organizations. After the spate of structural failures from the Hartford Civic Center through the Hyatt Walkways, a number of national conferences were held that were attended by experts on structural failure investigation and avoidance. Some of these conferences were:

- "Building Structural Failures—Their Cause and Prevention" (Santa Barbara, California, November 6–11, 1983)
- "Reducing Failures of Engineered Facilities" (Clearwater Beach, Florida, January 7–9, 1985)
- "Construction Industry Roundtable Meeting" (Kansas City, Missouri, March 1985)
- "Structural Failures II" (Palm Coast, Florida, December 6–10, 1987).

Out of the careful deliberations of each of these conferences came experts' recommendations for actions needed in the design and construction industry to mitigate the chances of catastrophic failure. The similarity among the recommendations that came from each conference is striking. All advanced the need for structural peer review.

American Consulting Engineers Council (ACEC)/American Society of Civil Engineers (ASCE) Guidelines. ACEC and its affiliated organization, the Coalition of American Structural Engineers (CASE), have developed several programs and guidelines for peer review. ACEC conducts Organizational Peer Reviews. In this type of review the quality of an engineering firm's professional practice is examined independent of a particular project through a quality audit. CASE has developed a comprehensive set of guidelines for Project Peer Review (CASE Document 5-1992).[5] That document contains not only a comprehensive suggested summary of services for peer reviews, but also provides recommended terms of agreement between the owner and structural engineer for project peer review services. ACEC and ASCE formed a joint task committee to develop a policy and set of proposed guidelines for project peer review. The resulting recommendations, ACEC Publication 1021, were introduced in 1990.[6]

Requirements in Connecticut. Public Act 89-255, which went into effect on July 1, 1989, requires an independent structural engineering review of large structures that exceed an established threshold limit. Excerpts from this legislation are reproduced below:

> "*Threshold Limit.* Any structure or addition thereto (1) having four stories, (2) sixty feet in height (3) with a clear span of one hundred fifty feet in width, (4) containing one hundred fifty thousand square feet of

total gross floor area, or (5) with an occupancy of one thousand persons.

"*Procedure.* If a proposed structure or addition will exceed the threshold limit as provided in this section, the building official of the municipality in which the structure or addition will be located shall require that an independent structural engineering consultant review the structural plans and specifications of the structure or addition to be constructed to determine their compliance with the requirements of the state building code to the extent necessary to assure the stability and integrity of the primary structural support systems of such structure or addition. . . . Any fees relative to such review requirements shall be paid by the owner of the proposed building project. The building official may prequalify independent structural engineering consultants to perform the reviews required under this subsection. . . . If fabricated structural load-bearing members and assemblies are used in such construction, the professional engineer licensed in accordance with chapter 391 responsible for the design of such members or assemblies shall sign a statement of professional opinion affirming that the completed fabrication is in substantial compliance with the approved design specifications."

While the scope of the required peer review is not as clearly defined as in the MSBC, the objective of the Connecticut requirements is similar. A professional organization, the Connecticut Engineers in Private Practice, has developed guidelines for the reviewing engineer.

Requirements in Other Countries. In West Germany "proof engineers" have practiced a form of project peer review for over fifty years. Design reviews by independent proof engineers are mandatory for major structures. The proof engineers are neither building inspectors nor project peer reviewers; they are federally licensed, independent peer consultants who are retained by municipalities. Proof engineers are licensed in three fields: metals, concrete and masonry, and wood construction. Their responsibility is to ensure that the design complies with all government regulations.[7]

For the owner to obtain building permits at the different stages of construction, several proof reports are required from the proof engineer. These reports verify the soil conditions, the architect's design and the structural integrity of the design. For complex structures, detailed checks of computations, drawings and any temporary support are also required. The cost of the review adds 0.6 to 1.0 percent to the construction cost, depending on the nature of the construction.[8]

In Belgium, the Bureau de Controle pour de la Securité de la Construction en Belgique (SECO) supervises all phases of design and construction. SECO is a nonprofit institution organized like an engineering consulting firm. It represents all Belgian insurance companies which, in turn, support it financially. When an owner seeks insurance for a proposed building, he or she submits the design to an insurance company. Before the insurance company will write a policy, SECO reviews the design. A technical board of 11 university professors arbitrates any differences of technical opinion between the designers and SECO's engineers. If the owner does not adopt the suggestions of SECO, the insurance company will not insure the building.[7]

France employs a peer review system very similar to the Belgium's SECO system.

Boston Second Engineer Check. On January 25, 1971, a 16-story apartment building known as 2000 Commonwealth Avenue in Boston collapsed while under construction. Four workers lost their lives. Investigations revealed a number of problems with the design, detailing and procedures used in the concrete construction. This building's failure prompted a program of project peer review in Boston. Many regulations for the design and construction of buildings for certain "Affidavit Projects" — one of them being a requirement for peer review of certain complex structures or systems — were formulated as a result of that collapse.

The threshold criteria for defining which buildings would be subject to peer review and the requirements of the review itself were not well defined. The Chief of Plans and Permits Division of the City of Boston Building Department was to determine whether or not the project was complex and subject to review. The

chief set forth official recommendations and the bases for such recommendations in a memorandum to the Commissioner of the Building Department, who could approve or disapprove the action. The commissioner further defined this criterion as follows:[9]

> For this purpose, a complex structure shall mean any building or structure whose construction cost is a million dollars or more [in 1971 dollars]. This is not a hard and fast rule, but it will serve to indicate to owners and designers that a second examination by a professional engineer is required by the building department.

The building owner could select the reviewer and underwrote the cost of the review. The examination could only be performed by an independent professional engineer or architect who was registered in the Commonwealth of Massachusetts.

After the examination, the reviewer would confer with the designer to report the findings. Any problems with the design were resolved to the mutual satisfaction of the designer and the reviewer. Thereafter, the reviewer would prepare a letter to the commissioner stating that the reviewer had "checked the details, computations, stress diagrams, and other data necessary to describe the construction and basis of calculations, and further stating that, in [the reviewer's] judgment, the requirements of the City of Boston Building Code have been met with respect to the design."[9]

The required procedures and scope of the review process were similarly vague. Again, the commissioner elaborated:[9]

> I expect [the reviewer] to make a determination that the foundation design will support the structure. [The reviewer] should check the building for dead and live loads, and spot check the designer's assumptions, and check a percentage of the calculations, using something in the order of five or ten percent of these. . . . When the examining engineer has completed [the] review of the design of the structure, I want [the reviewer] to write a letter to the building department to the effect that [he/she] has reviewed the design based upon a spot check of some of the critical parts of the design saying something to the effect, "Based upon a spot check, I believe that the structure is designed in accordance with the provisions of the City of Boston Building Code."

Reaction to the Boston project peer review procedure was mixed. Some felt it was worthwhile and some did not. The perceived variability in success seemed to be due to a lack of clarity in the review regulations and the fact that owners could select reviewers largely on the basis of fee. The Boston procedure was superseded by the MSBC requirements in 1992.

The Massachusetts Requirements

The proposal to develop mandatory state-wide requirements for structural design peer review was initiated by the Chairperson of the Board of Building Regulations and Standards (BBRS). The chair became concerned over the increasing seriousness of reports of non-conforming seismic design practices in new structural design. The chair brought his concerns to the Boston Association of Structural Engineers (BASE). In considering the chair's request, BASE judged that non-conforming design was a concern for all life-safety aspects of structural design, not just earthquake resistance. As a result, BASE developed a comprehensive proposal for peer review, which was first submitted to the BBRS in February 1991.[10,11] The Boston Society of Civil Engineers Section/ASCE and ACEC acted as BASE's sounding board by commenting on draft versions of the document. The BBRS adopted the BASE recommendations with some modifications, and the requirements for peer review became effective on June 19, 1992.

The purpose of the MSBC requirements for project peer review is to enhance public safety of new construction. Toward this goal, the peer review should "verify that the design of primary structure is conceptually correct and that there are no major errors in the design" [Article 1, Par. 113.8.].[12]

Not all structural designs must be reviewed, only those where risk to public safety — because of size or occupancy of the building — is substantial. Structures for which the MSBC requires review are:

- Buildings that are five stories or more in height above the lowest floor, including stories below grade.
- Buildings that enclose a total volume of 400,000 cubic feet, including stories below grade. The volume is measured using the outside dimensions of the building.
- Structures in Use Group A, or structures that are partially in Use Group A, that will be used for the public assembly of 300 or more persons for civic, social or religious functions, recreation, food or drink consumption, or awaiting transportation.
- Structures of unusual complexity or design as determined on a case-by-case basis by the BBRS. A building official may apply to the BBRS for such a determination on a specific structure.

The only exemption is granted to temporary structures that are to be erected for a period of 180 days or less.

The MSBC requires that the owner retain the reviewing engineer. Mandatory qualifications for the reviewer are as follows:[12]

> The reviewing engineer shall be a professional structural engineer, registered in Massachusetts, qualified by experience and training and who shall have had structural design experience with buildings or structures similar to that covered by the application for the building permit. The reviewing engineer shall be impartial, and shall be independent of the architect of record, structural engineer of record, and contractors and suppliers who will be involved in the construction of the structure. [Appendix I, Par. I-2.1]

The reviewing engineer verifies that the plans and specifications submitted with the application for the building permit complies with the structural and foundation design provisions of the MSBC. The review must entail the following activities:

- Check to assure that the design loads conform with the MSBC;
- Check that other design criteria assumptions conform to the MSBC and are in accordance with accepted engineering practice;
- Review geotechnical and other engineering investigations that are related to the structural design to determine if the design properly incorporates the results and recommendations of the investigations;
- Check that the organization of the structure is conceptually correct; and,
- Make independent calculations for a representative fraction of systems, members and details to check their adequacy. The number of representative systems, members and details should be sufficient to form a basis for the reviewer's conclusions.

The reviewing engineer is not obligated to obtain and check the design calculations of the structural engineer of record, but may request a copy of these calculations for reference. If the design criteria and assumptions are not stated on the design drawings or in the project specifications, the structural engineer of record must set these forth in writing to the reviewer.

The MSBC requires a report and follow-up as indicated below:

- The reviewing engineer shall prepare a report to the building official stating whether or not the structural design shown on the drawings and the specifications conforms with the structural and foundation requirements of the MSBC. That report should include a summary of all deficiencies, if any, that cannot be resolved with the structural engineer of record.
- The structural engineer of record must review the report of the reviewing engineer and notify the building official in writing, whether or not there is any agreement with the conclusions and recommendations of the reviewing engineer.
- Unresolved disputes between the structural engineer of record and the reviewing engineer should be submitted by the building official, the owner, the structural engineer of record or the reviewing engineer to the Structural Peer Review Advisory Board for resolution.
- Any changes to the structural design subsequent to the original submission of the plans and specifications should be shown

on revised drawings and specifications, submitted with an amendment to the application for permit. The reviewing engineer must review the changes on the revised drawings and specifications. If the original report does not account for the changes in those drawings and specifications, a supplementary report relating to the changes and prepared by the reviewing engineer should be made to the building official.

The MSBC allows for special review and approval of the permits to construct foundations. Sufficient documentation must be made available so that the reviewing engineer can review the criteria and structural concepts for the whole structure as well as be provided with complete independent calculations for the part of the foundation covered by the permit.

Procedural & Professional Issues

There are some misunderstandings among building owners and others regarding the purpose of the Massachusetts peer review requirements. The peer review:

- Is *not* an exhaustive check of the structural design;
- Is *not* intended to mitigate the occurrence of minor failures or serviceability problems (*e.g.*, excessive vibrations or deflections);
- Is *not* intended to identify potential cost savings in the design;
- Does *not* involve elements of the building other than the principal structural system; and,
- Is *not* an opportunity for the building owner to engage the liability insurance of another engineer.

Scope & Cost. While the MSBC sets forth five tasks incumbent upon the reviewer (as stated in the previous section here), there is still substantial latitude and judgment given within the scope of review. Consequently, the thoroughness and effectiveness of the review can vary. Peer reviews by very experienced engineers can be very effective and efficient means of spotting gross problems. A seasoned eye is invaluable. The wise owner will select a reviewer largely on the basis of experience and qualifications rather than fee. There are no guidelines for the cost of a peer review, but where the review does not involve the resolution of an unusual incidence of problems, it can usually be performed for a small fraction of the design fee of the structural engineer of record.

Timing. The provisions of the MSBC require that the plans and specifications for the building permit be reviewed. Accordingly, most peer reviews are usually performed near the completion of the structural design (around the 90 percent design stage). The BASE Commentary recommends starting the peer review at the end of the preliminary design phase.[11]

The timing of the review must be carefully considered. If the review is performed too late in the process, construction can be delayed and the correction of any deficiencies can be expensive. A building permit is not issued until the review has been completed and any outstanding issues have been resolved. If disputes cannot be resolved between the engineer of record and the reviewing engineer, then the dispute is brought before a Structural Peer Review Advisory Board. Such arbitration can further delay a project.

Very large projects benefit from early peer reviews of criteria and concepts performed at the end of the design development phase. Detailed checking of member adequacy is performed once the construction documents are well advanced. There is a small fee premium to perform an early-start peer review, but it is generally cost effective for large or complex projects, since changes can readily be effected at that stage.

Dispute Resolution. Disputes may arise in the review process over differences of opinion between the reviewer and the structural engineer of record regarding the adequacy of one or more particular elements of the design. Recognizing this possibility, the MSBC allows for the mediation of such disputes by the Structural Peer Review Advisory Board.

The structural engineer of record, the reviewing engineer or the owner of the building submits any unresolved disputes cited to the Structural Peer Review Advisory Board on a form provided for this purpose. The board then convenes a mediation hearing within 30 days

of receiving the application. It must render a decision in writing within 30 days following the mediation hearing.

In spite of having an established procedure for redressing disputes, the profession and all involved parties can be best served when the engineer of record and the reviewer establish a direct line of communication and a working relationship that can be used to resolve such disputes without intervention. Each party must approach this type of working relationship with the highest degree of professionalism and with a spirit of cooperation. The owner need not be involved in, or apprised of, the content of such technical discussions unless they impact schedule, construction budget, engineering fees, or some other direct concern of the owner. If the reviewer and the engineer of record cannot come to an agreement within this framework, the owner may direct that the more conservative opinion be adopted, or the owner may engage a third engineer to help resolve the dispute.

Responsibility & Liability. The MSBC states that:

> The structural engineer of record shall retain sole responsibility for the structural design, and the activities and reports of the reviewing engineer shall not relieve the structural engineer of record of the responsibility.

It is appropriate that a single party retain sole responsibility for the design — to do otherwise would obfuscate the lines of responsibility and present opportunities for mistakes. CASE Document 5-1992 presents recommended terms for contracts between owners and reviewing engineers that appropriately limit the liability of reviewers.[5]

Qualifications of the Peer Reviewer. Technical qualifications and experience are not the only important qualifications of a peer reviewer. Also paramount are high standards of professionalism and skill in resolving disputes. From an owner's perspective, perhaps the greatest risk that peer review presents to the project is the possibility of a protracted mediation process caused by differences of opinion between the engineer of record and the reviewer. Strong communication skills, a willingness and ability to understand alternative points of view, and a commitment to a successful review process are key qualifications. In selecting a reviewer, an owner should look for an experienced and knowledgeable engineer with a demonstrated record of conducting meaningful and effective reviews.

Segmented Design Process. Special problems are introduced when a segmented design process (that releases various portions of the design for construction in a fast-track mode) is adopted. Sufficient documentation must be made available so that the reviewing engineer can examine the criteria and structural concepts for the whole structure as well as the complete independent calculations for the part of the foundation covered by the permit.

Opportunities Lost. Peer review cannot ensure that a structural design will be fully free from errors — not only because the review is less than exhaustive, but also because design problems can go undetected for other reasons. For example, it is common for contract documents to delegate the design of some elements of the structure to specialty subcontractors through performance specifications. Examples are post-tensioning design, precast concrete design, structural steel connections, and steel bar joists. Such specialty contractor design is typically performed after the peer review has been completed, so such elements are not subjected to peer review. Gross problems with specialty contractor designs led to some of the severe structural deficiencies at the L'Ambiance Plaza Apartments. Although the MSBC peer review provisions do not require it, such specialty contractor designs should also be subject to peer review.

It also is common for some parts of the structural design to change during construction, again some time after the peer review has been completed. While MSBC peer review requirements dictate the resubmission for review of any revised plans in such circumstances, this resubmission is infrequently done.

When Major Changes Are Required. On occasion, the reviewing engineer may encounter a structural design that is so grossly riddled with errors that the reviewer has no reason to have confidence that those elements that were not

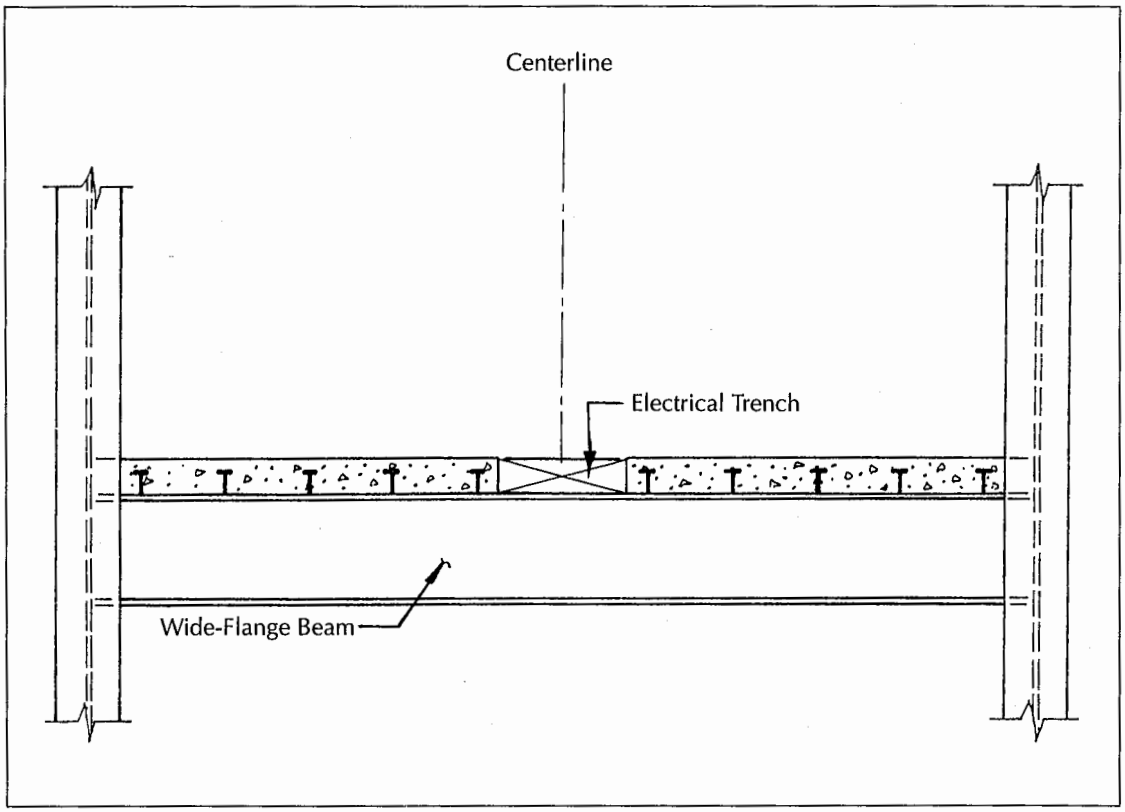

FIGURE 1. Composite action is destroyed by an electrical trench at mid-span.

reviewed could be reasonably free from errors. Consequently, the reviewer may feel that he or she cannot prepare a report in good faith representing that the structural design conforms to the MSBC. This situation dictates major corrective action be undertaken for the project. Such corrective actions can have severe financial and time consequences on the project, presenting a dilemma to the reviewing engineer for which there is no guidance in the MSBC. Remediation of the design in this case may involve one or more of the following:

- The structural engineer of record agrees to perform a full design check to correct all deficiencies, and resubmits the design for a new review.
- The peer reviewer is directed and paid by owner to make a full, exhaustive design check.
- A third engineer is retained to fully review the design and identify all areas needing correction.

Case Studies

A number of examples illustrate the benefits of, and types of problems exposed by, a peer review.

Office Building. The peer review for this six-story high, approximately 200 by 300 feet in plan, structure was not performed according to the MSBC peer review provisions, but was initiated by the owner after a partial failure occurred early in construction. The review of the structural design of this steel-framed structure revealed a number of problems.

The steel floor beams were designed to act compositely with the concrete slabs. Due to poor coordination between the structural and the mechanical/electrical drawings, an electrical trench was located perpendicular to the beams near midspan of the beams, essentially destroying composite action and greatly comprising the strength of the beams (see Figure 1).

Beam-column connections at the roof level were designed to be moment resisting, and the

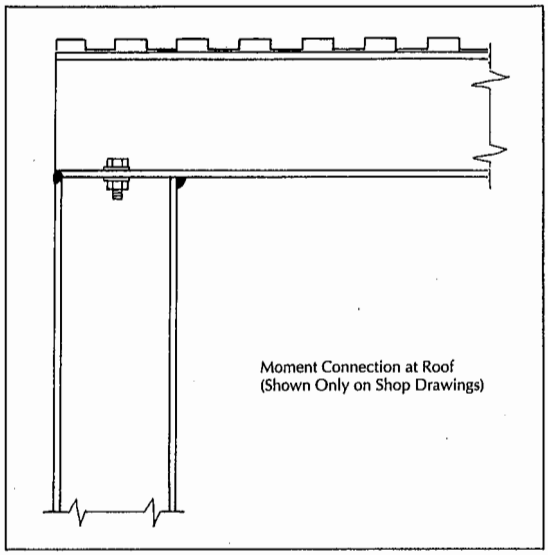

FIGURE 2. No stiffener plates were provided at the connection to transfer moment.

connections required stiffener plates to transfer the design moments. However, the connection design did not provide such stiffeners (see Figure 2).

A column adjacent to the stairwell opening in the floor slab was left unbraced by the slab opening. The column had no appreciable resistance to wind load (see Figure 3).

The building's structural system for resisting lateral wind and seismic loads consisted of moment-resisting frames, but not all of the columns had moment connections to allow them to participate in the lateral system. Consequently, the bracing required for the non-participating columns placed additional P-Δ forces on the participating columns. The building's original structural design did not consider P-Δ effects at all (see Figure 4).

Shopping Mall. This case study describes a peer review that was not performed according to the MSBC peer review requirements, since the project was located in another state. The peer review commenced at the beginning of construction.

The project involved a substantial renovation and addition to an operating retail mall. A small part of the work involved adding a cantilevered steel and concrete walkway to the side of an existing concrete wall. The walkway ran parallel to the wall. Shortly after construction of the walkway, the contractor and owner noticed that the walkway sagged over the length of its cantilever, and the connections of the walkway beams to the concrete wall were pulling away from the wall. The owner retained an independent reviewing engineer to investigate the problem. That investigation, as well as an investigation by the engineer of record, revealed that the cause of the walkway distress was a design error in the bracket connections that anchored the walkway support beams to the wall. The brackets were designed for shear, but the designer neglected the tension induced in the connection by inclined support struts that propped the ends of the walkway beams (see Figure 5).

Before implementing repairs for these failed walkway connections, the owner directed the

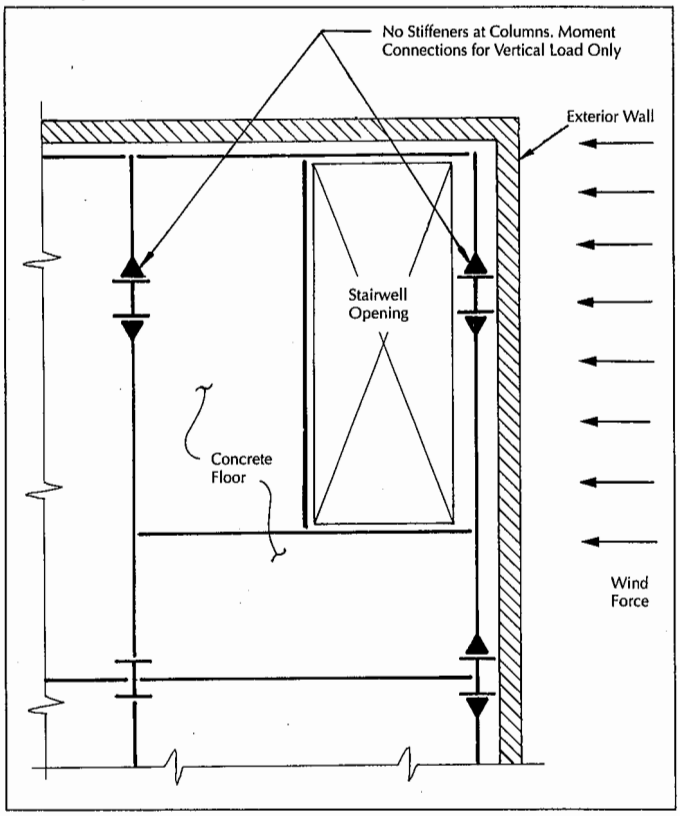

FIGURE 3. An unbraced column at the stairwell opening.

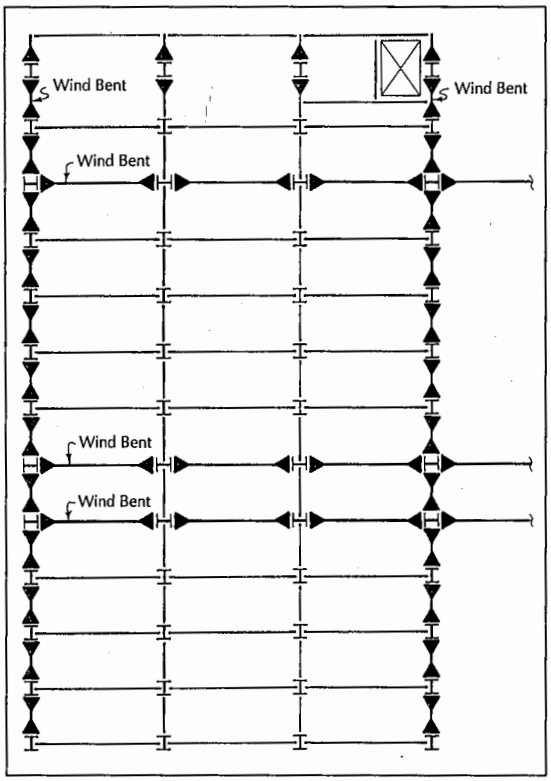

FIGURE 4. Only three of the 12 transverse frames have moment-resisting joints for resisting lateral wind and seismic loads.

structural designer and the reviewer to independently check the design of the other elements of the walkway. These checks revealed several other problems, causing the owner concern over the quality of the structural engineer's design of the mall renovation in general. At that time, construction of the project (worth tens of millions of dollars) was in an early phase, but fully underway. Any errors revealed in the structural design could have had a major impact on the project, and time was urgent. Because some of the construction had been completed, some remedial work would have to be performed on work in place. In other areas it was not too late to change the design. The owner, the structural engineer and the reviewer agreed to the following plan:

- The structural engineer would review the entire design and identify any necessary remedial work.
- The reviewer would check two representative areas of the structure comprising about 10 percent of the project.
- The reviewer was under instructions not to reveal to the structural engineer of record which areas would be reviewed.
- All agreed to meet after all assignments had been completed to compare results.

After a month all parties met. The structural engineer had found several additional design errors. The reviewer found many more errors in the representative areas the reviewer checked. A few are mentioned below. These problems were not necessarily the most severe, but are interesting for the challenges they presented.

The design of the roof trusses supporting sloped skylight areas of the main mall is shown in Figure 6. The single angle web members were connected to the chords with single bolts. Most of the trusses were fabricated and some were erected at the time of review. The peer reviewer felt that the use of single bolt connections in this application, especially considering the eccentricities involved in the connections, was imprudent. The structural engineer believed that the connections were adequate. All agreed to resolve this problem by welding the

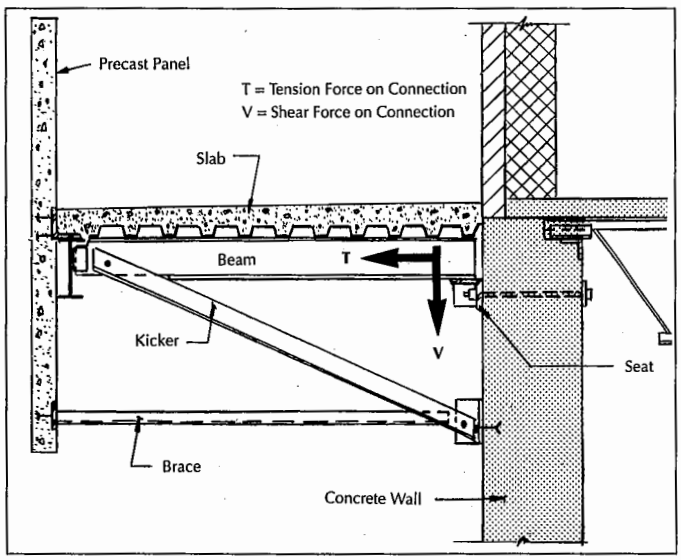

FIGURE 5. The design of the beam seat ignored the tension force imposed by the inclined strut.

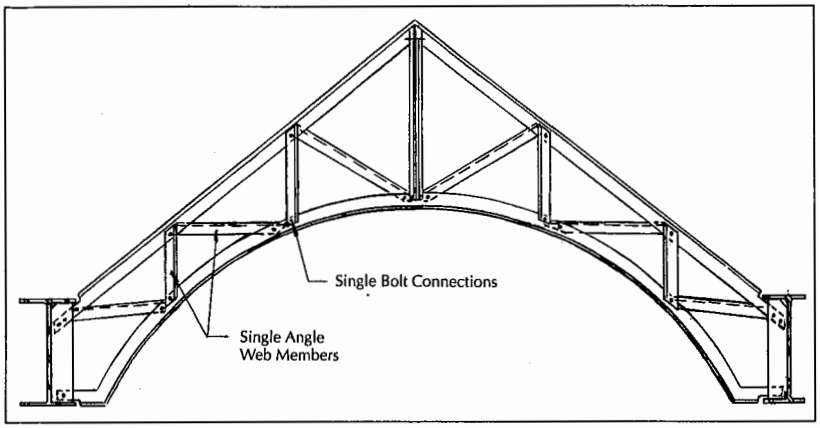

FIGURE 6. Typical roof truss for the shopping mall project.

most highly stressed of the single bolted connections near the ends of the trusses.

One unusual feature of the structure was a set of large space trusses that spanned open court areas of the mall as shown in Figure 7. The primary gabled arch trusses were supported on ring trusses forming the perimeter of the opening. The ring trusses were supported, in turn, on columns. For architectural reasons, the ring trusses were unusually deep compared to their span, making them quite stiff. As a consequence of this unusual stiffness, the structure was very sensitive to differential foundation settlements at the supporting columns. Small foundation settlements overstressed several of the truss members. The structural engineer did not consider the effects of foundation settlement in the original design.

Another problem that was revealed is shown in Figure 8. In order to provide raised skylight areas over parts of the existing mall roof, the existing roof bar joists were to be cut and suspended from new longitudinal support beams on each side of the skylight openings. There were several problems with the design of the connections of the truncated joists to the new beams, caused principally by the eccentricities and the lack of lateral stability of the connections.

As a result of the multitude of design problems that the peer review uncovered, the owner lost confidence in the original structural design. However, the owner was reluctant to relieve the engineer from the project, given the fact that construction was well underway. The owner commanded the peer reviewer to per-

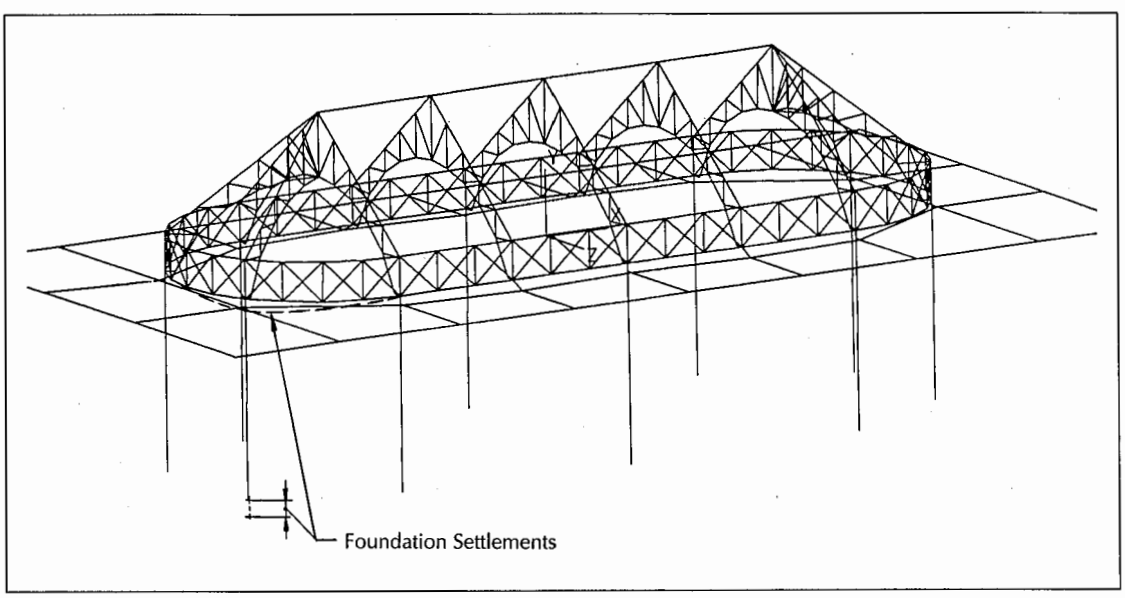

FIGURE 7. Small foundation settlements overstress truss members.

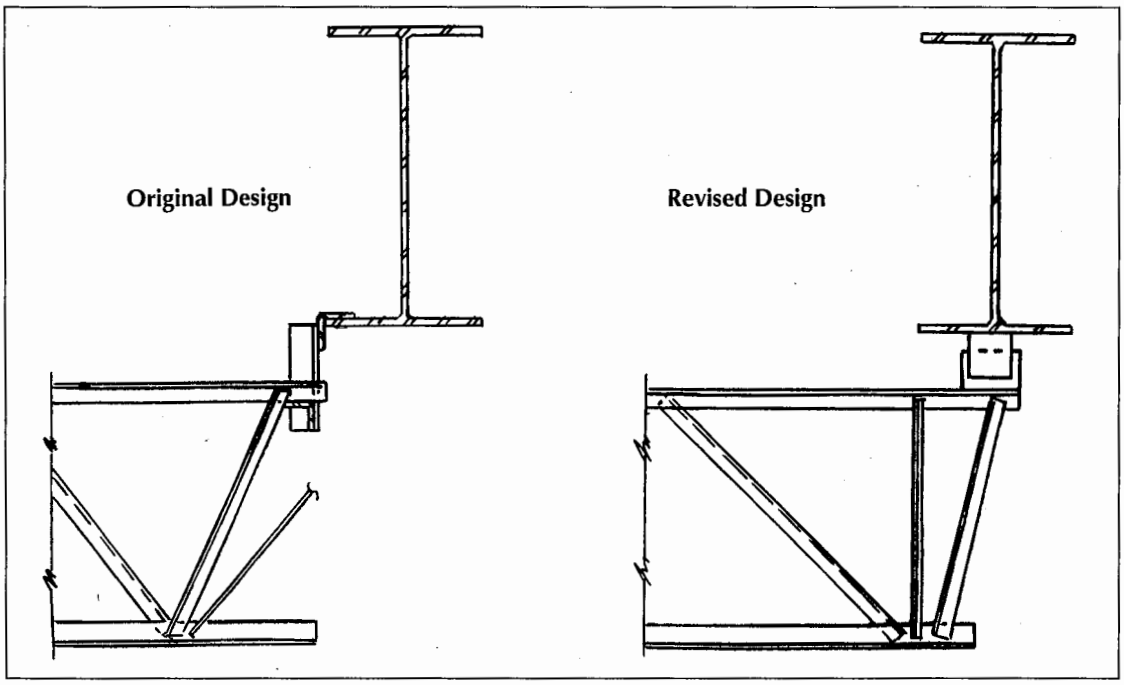

FIGURE 8. Connections were redesigned to eliminate eccentricities and lack of lateral stability.

form a complete design check of the work, which was done. This full design check revealed further problems that were corrected as the work proceeded.

There were other problems with the quality of the structural work on this project due to poor construction techniques, critical demolition procedures that were not engineered or otherwise thought out, ineffective inspection and lax submittal procedures. Much of these problems stemmed from a lack of clearly defined responsibility and lines of communication on this very complex construction project. The owner, construction manager and peer reviewer made a thorough review of, and revision to, the quality plan early in the construction process. In spite of all of the problems, the project was completed on time.

Hospital Building. This six-story hospital building was a reinforced concrete structure (approximately 270 by 150 feet in plan) braced by shearwalls. The structure was designed to accommodate three future stories. A number of problems were revealed by the peer review.

Cantilevered grade beams were required to support ten levels (nine stories and a roof). The design incorrectly considered the top steel to be fully effective in providing shear reinforcement for the cantilever portion of the grade beam rather than providing additional uniformly-spaced horizontal reinforcement. The top steel was not fully effective due to the lap splices that were detailed (see Figure 9).

The designer's lateral load distribution to the shear walls was incorrect. The shear walls were designed only for tension forces, neglecting very large compression forces. The shearwall reinforcement of the original design called for 15 #11 bars with no ties. After correcting the errors, the shearwall reinforcement was 63 #11 bars with ties (see Figure 10).

The typical floor framing consists of 10-inch thick two-way flat slabs with a dead weight of 125 pounds per square foot (psf) and 16-inch one-way slabs with a dead weight of 200 psf. There are 3.5-inch slab depressions in the 10-inch two-way slabs. The designer did not consider the additional weight of 75 psf for the 16-inch slabs or the additional 40 psf for finishes in the 3.5-inch slab depressions. The dead load was underestimated by 25 to 50 percent for the design of some columns. The dead load was

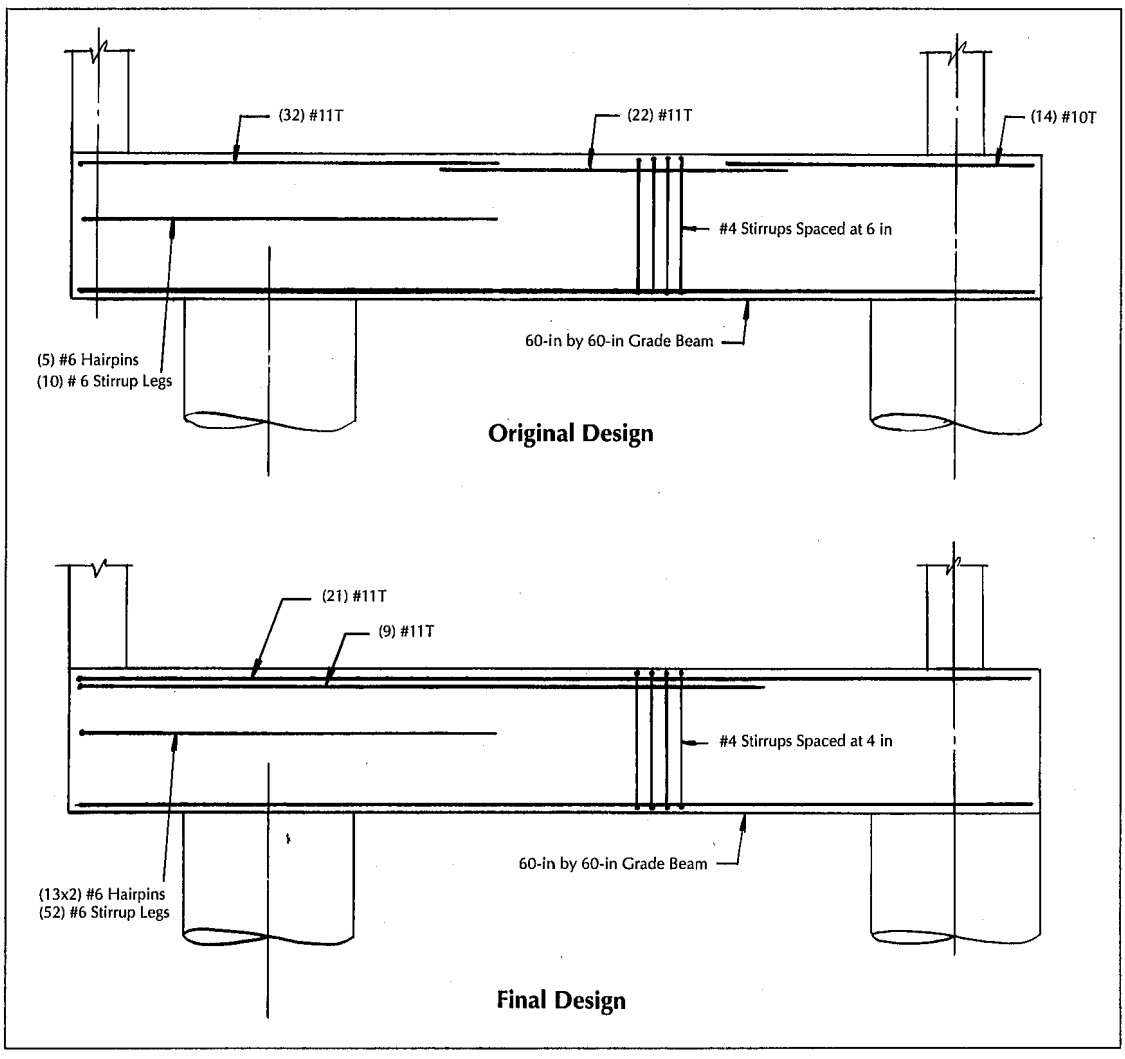

FIGURE 9. Cantilever grade beams were redesigned to correct inadequate stirrups and lap splices.

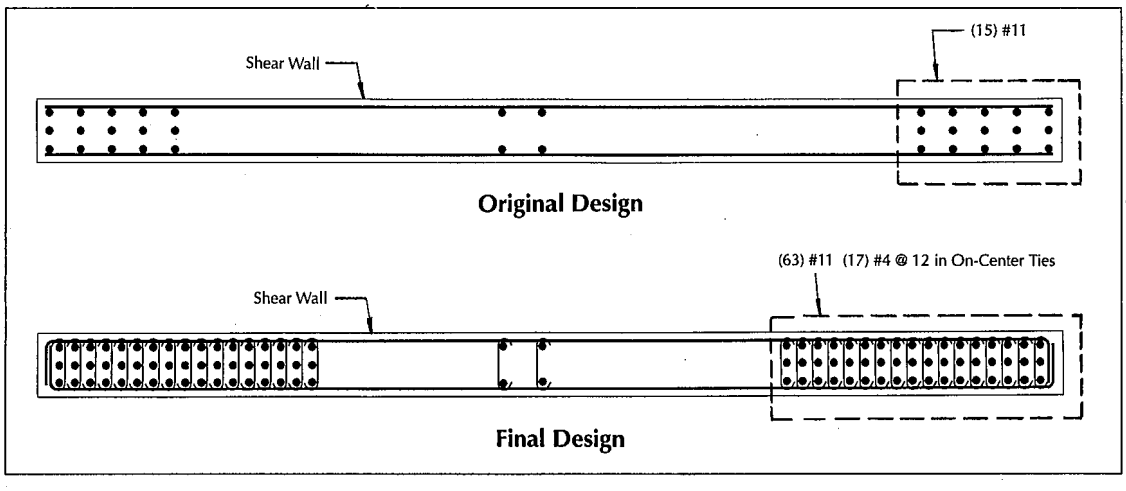

FIGURE 10. Shear walls were redesigned to account for compressive forces.

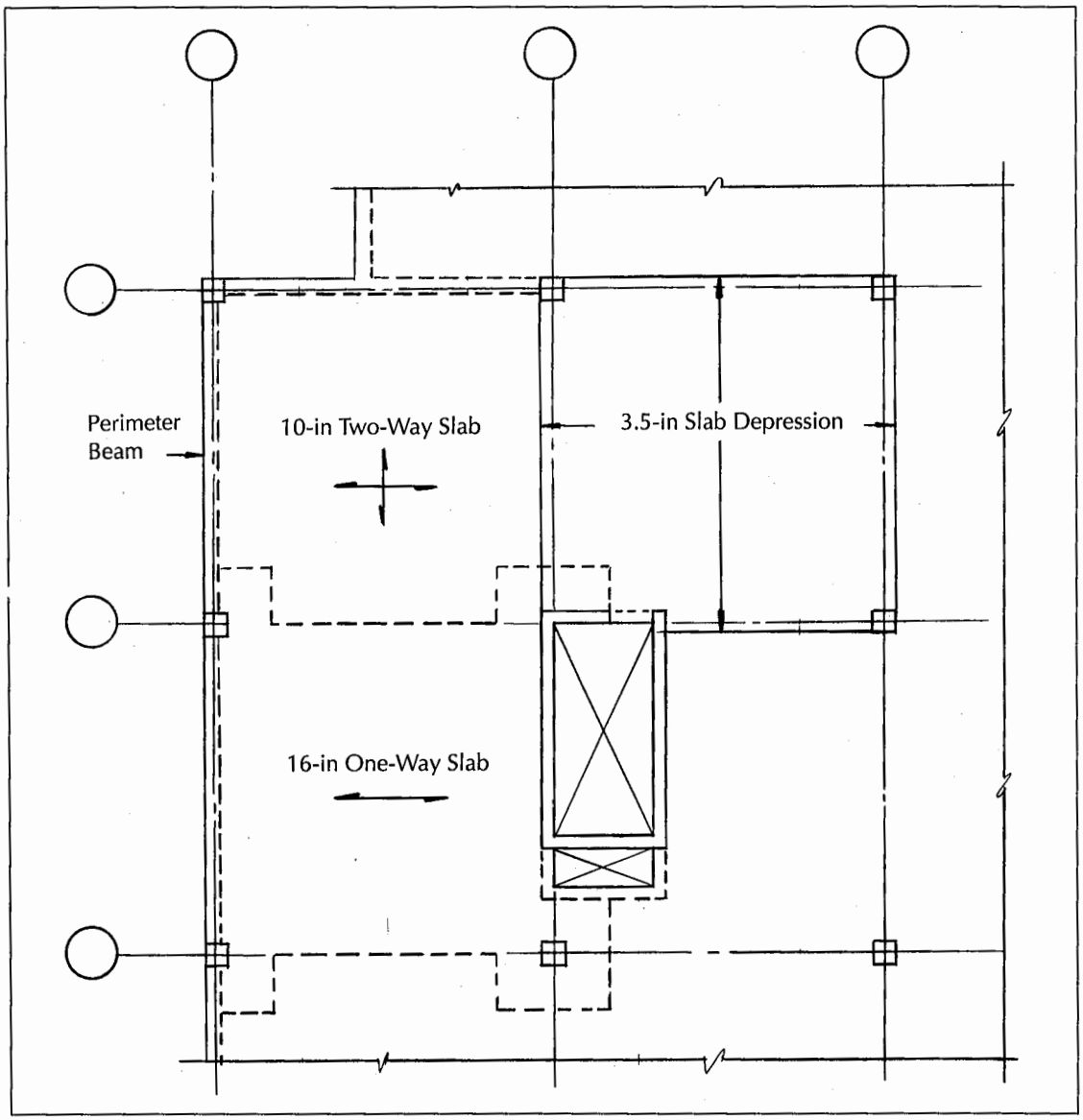

FIGURE 11. Typical floor plan for the hospital project.

underestimated by 25 percent for the design of the floor slab at the depressions (see Figure 11).

Caissons supporting shear walls were required to resist large tension loads (2,000 to 3,000 kips) with large shear loads (100 to 300 kips). The analysis of the caissons for shear was incorrect, and the design did not account for the reduction in the allowable shear capacity to account for tension. Additional stirrups were required in the area of high shear starting from the surface of the rock to several feet below. Shear studs on the steel embeds to transfer tension loads were missing in the lower half of the caissons (see Figure 12).

The six-story structure was supposed to be designed to accommodate three additional stories in the future using the roof as a story as shown on architectural drawings, but the structure was actually designed for only two future stories. This error was caused by miscommunication between the project engineer and a designer not familiar with the design criteria for the project. The project engineer directed the designer to design the structure for two future

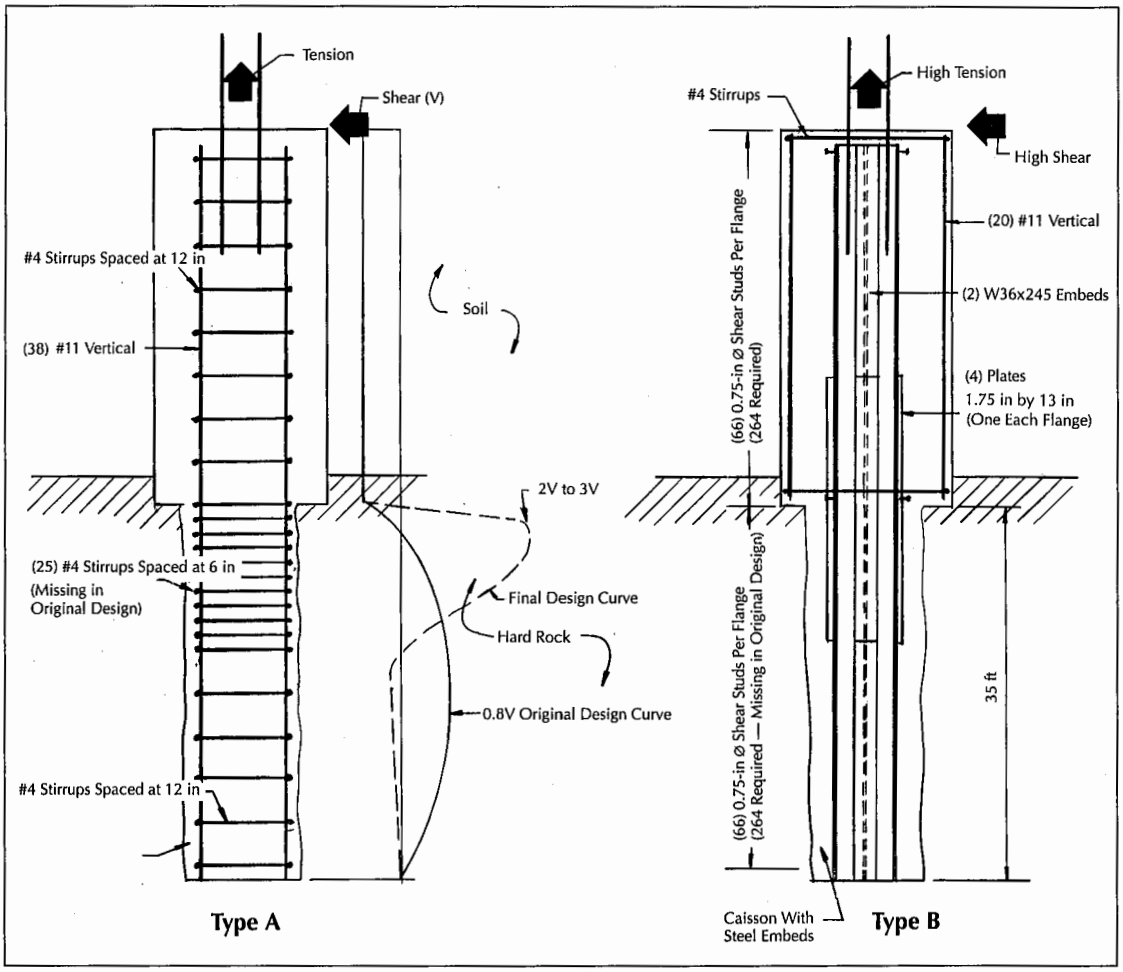

FIGURE 12. Caissons were redesigned to account for tension forces neglected in the original design.

stories. The designer was not aware that two additional framed levels and a roof level were required to accommodate three future stories and did not look at the architectural drawings. All the columns and shearwalls were redesigned to support the weight of the missing story.

A large combined foundation mat was designed to resist the overturning imposed by three shear wall stair towers. The design analysis did not consider the unsymmetrical shape of the mat and significantly overestimated the overturning resistance of the mat. After the peer review the mat was redesigned to include 60- by 90-inch continuous grade beams spanning to adjacent columns to provide additional overturning resistance (see Figure 13).

Conclusions

While Massachusetts is one of the first states to adopt mandatory structural peer review for threshold structures, the need for peer review is receiving more consideration nationwide. The need is based on the recognition that the complexity of current design codes and specifications, the complexity of modern design and construction teams, the use of higher performance structures with lower factors of safety, and the severe business and time pressures increase the probability of catastrophic life-threatening failures. A rash of catastrophic building failures since the early 1970s demonstrates that this concern is valid.

Peer review can be effective in reducing errors in new structural designs, but the process

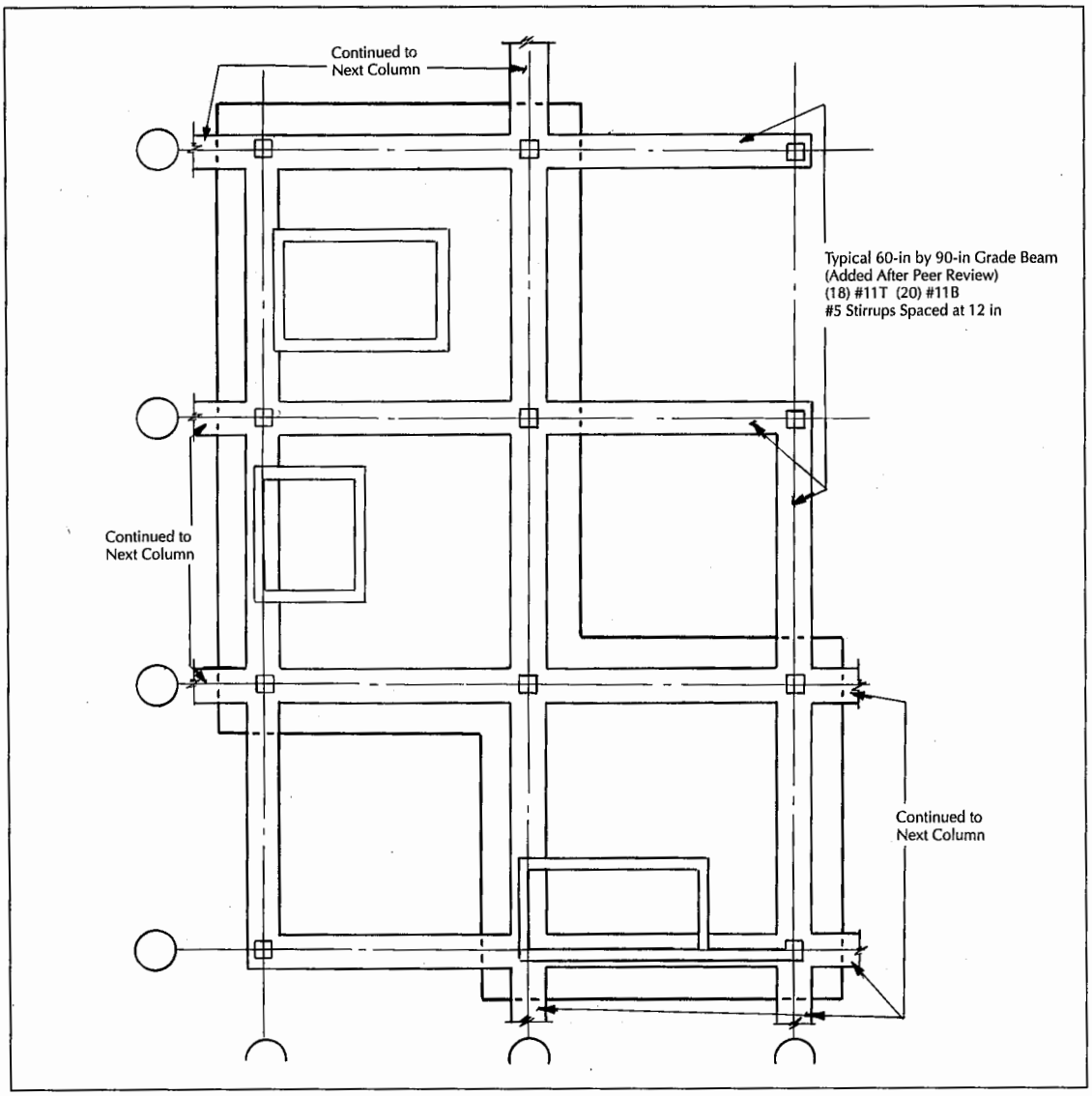

FIGURE 13. Plan of the mat foundation for the hospital building.

poses several challenges in execution. The peer reviewer must be selected not only for technical competence and experience, but also for skills in problem resolution. The scope and timing of the review also can be critical to the project's success. Segmented design processes and changes to the design during construction present opportunities for errors to go undetected by the review.

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