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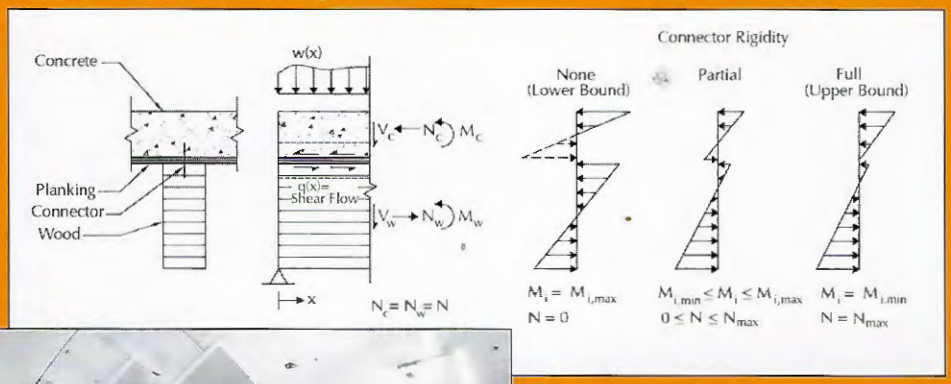
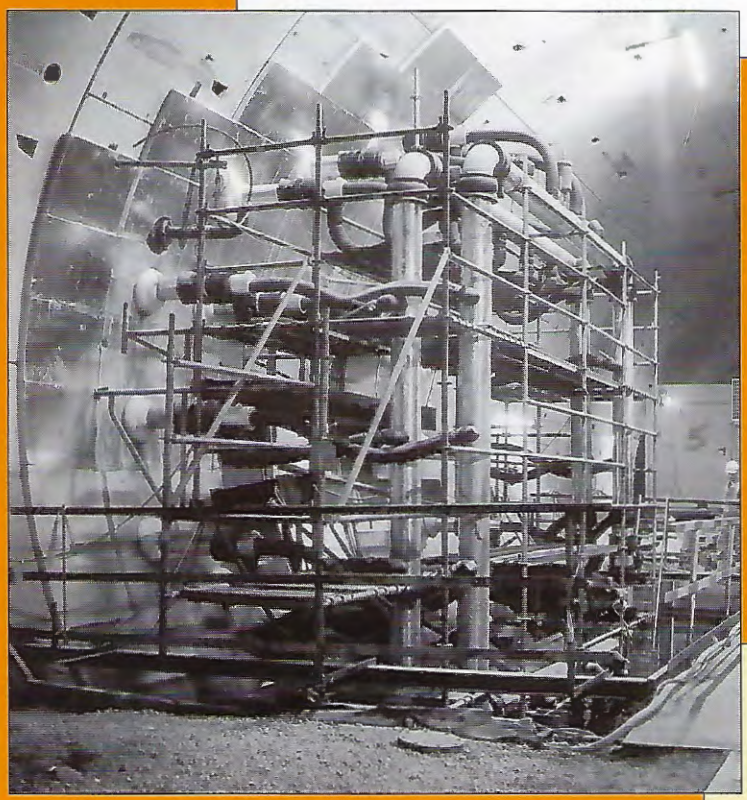
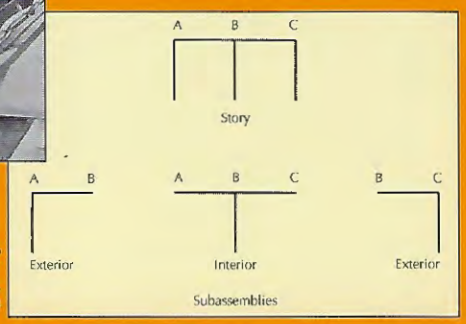
Wood-Concrete Composites





Using Ground Freezing for Tunneling



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Wood-Concrete Composites: A Structurally Efficient Material Option

The use of wood-concrete composite systems for new commercial and residential floor construction, as well as for renovations, can provide distinct advantages.

PEGGI CLOUSTON & ALEXANDER SCHREYER

Wood-concrete composites (WCCs) are composite deck structures that consist of a solid cast-in-place concrete slab that is placed upon, and integrally connected to, wood members below. Although not commonly seen today, WCCs have been used since the early 1930s in North American timber bridge construction. The two most prevalent types are the T-beam deck (see Figure 1) and the composite slab deck (see Figure 2). Traditionally, the shear connection between the wood and concrete was formed through slots cut into the wood members to form concrete shear keys or by mechanical fasteners driven into the top of the wood, or both. Recent research and development efforts throughout the world have advanced the technology and understanding of WCCs and have, consequently, increased the interest and acceptance of these systems in North America of late.[1–5]

The concept of a composite deck is not novel — it is most commonly associated with steel-concrete composite construction. The two materials act in unison and, through composite action, the material achieves overall stiffness and strength that is superior to that of either of the components acting alone. In the case of a wood-concrete composite, the concrete slab (having virtually no tensile capacity) is used predominantly in compression where it has superior performance in terms of strength and stiffness. The wood (in the form of glue-laminated timber or structural lumber) is used predominantly in tension, which provides exceptional strength and stiffness and results in reduced weight when compared to an equivalent all-concrete section. The end result is a strong, rigid and relatively lightweight deck.

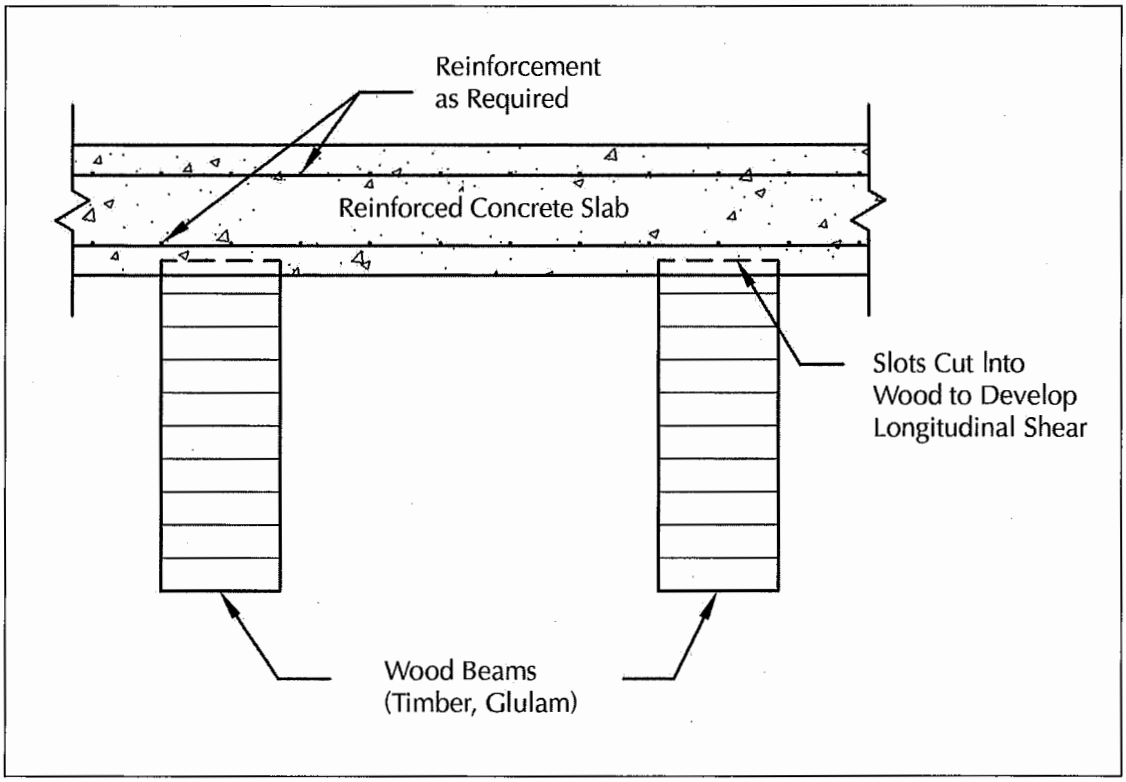

FIGURE 1. A wood-concrete composite T-beam deck.

A significant market for the WCC has been established in restoring and upgrading historical buildings in Europe, where WCCs are considered "an economical and ecological alternative to removing the old floor and replacing it with a reinforced concrete slab."[6] Figure 3 shows one such project — a structural upgrade for an attic in an historic home in Aschaffenburg, Germany. WCCs are used to improve serviceability (i.e., deflection and vibration). They are also used to upgrade the load capacity of old timber floors. The renovation process involves installing metal shear connectors in the wood members and pouring a cast-in-place concrete slab over the existing timber floor. The floor (typically timber beams with transverse planking) becomes permanent formwork. Temporary shoring is generally used to support the weight of the wet concrete prior to curing and achieving composite action. With this process, construction times are quick and historical ceilings are preserved.

Similar opportunities exist for the application of WCCs in the United States in the restoration of old mills and other industrial buildings (particularly in southern New England). General interest for reusing old structures (quite often with timber flooring) is on the rise as evidenced by local news articles (such as in the *Boston Sunday Herald* and the *Norwich Bulletin*[7,8]) and feasibility studies (for example, by Schreyer[9]). Many of these abandoned buildings are ideally located next to a river or in a downtown area and have excellent potential for upgrade to residential or commercial use.

There are also reasons to consider WCCs for new construction. For instance, for commercial floor systems — housing units, schools and public buildings — the improvement of sound and vibration performance as a result of the added mass of the concrete offer distinct advantages. WCCs also perform well in terms of fire resistance, with an achievable rating of up to 90 minutes.[10] Concrete has a natural high resistance to fire, and large timbers form a char that naturally insulates both the timber core and any metal shear connec-

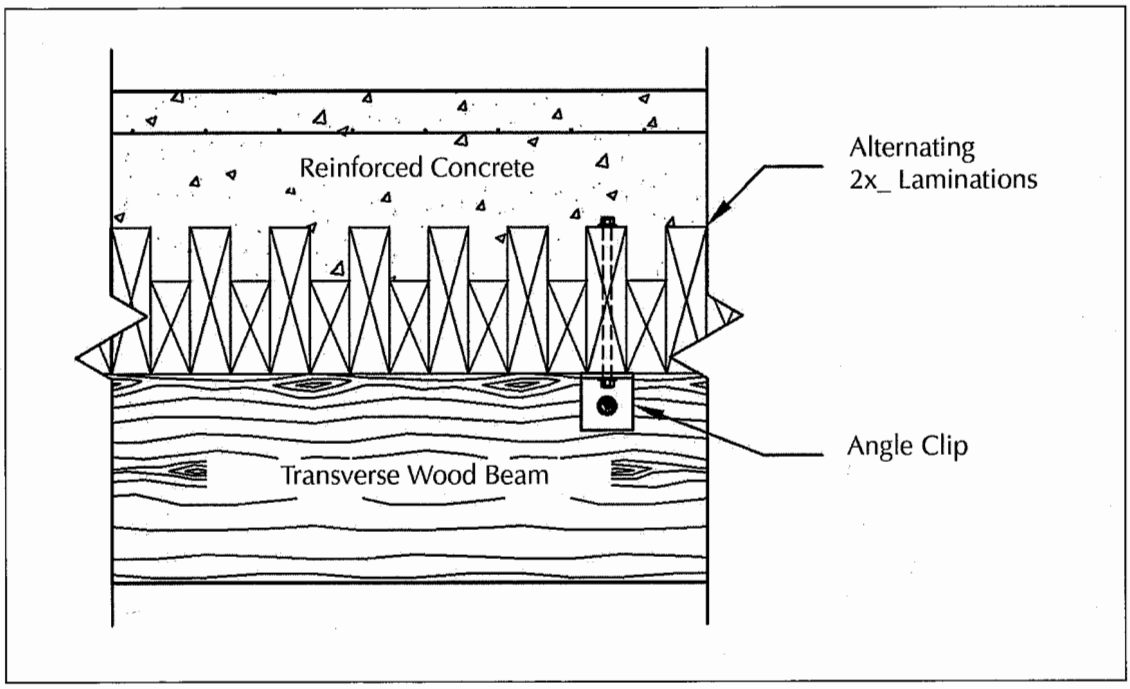

FIGURE 2. A wood-concrete composite slab deck.

tors against high temperatures. In the use of pedestrian or vehicular bridge decks, durability is improved. A fully exposed wooden bridge deck will deteriorate from exposure to weather as well as from common wear and tear. In a wood-concrete composite, the concrete slab is able to protect the wood beams beneath by providing a wear-resistant surface from which water can run off. Conceivably, preservative treatment for the wood (and the related strength loss as well as detrimental effects on the environment) could be avoided if the wood members could be reliably protected from weathering.

From a sustainability standpoint, WCCs are superior to reinforced concrete or steel options due to the environmental merits of wood. In addition to recognizing that wood is the only primary building material that comes from a renewable resource, it is also the lowest in energy requirements for its manufacturing and life-cycle assessment. Recent studies by university and industry research groups in the United States (by the Consortium for Research on Renewable Industrial Materials [CORRIM]) and in Canada (by the Athena Sustainable Materials Institute) have indicated that wood structures possess the least embodied energy (energy used to acquire raw materials as well as to process, manufacture, transport and construct) when compared to similar structures of concrete or steel. Wood also consumes the least amount of operating energy (energy used for heating, cooling, lighting, etc.). Both embodied and operating energies mainly use nonrenewable fossil fuels, which release deleterious greenhouse gases (such as carbon dioxide and nitrous oxide) into the environment. Buildings account for approximately 40 percent of primary energy consumption in the United States.[11] The need to reduce this consumption is widely recognized. Even a modest increase in the use of wood through WCC construction could help reduce the impact of building on the environment.

The primary drawback to some WCC systems is the added construction time and cost required to prepare and/or install the shear connectors. Cost advantages are found, however, in labor savings by using the timber as the permanent formwork, using less material for foundations as a result of lower floor dead loads (wood being lighter than concrete or steel), and, in the case of restoration, quicker

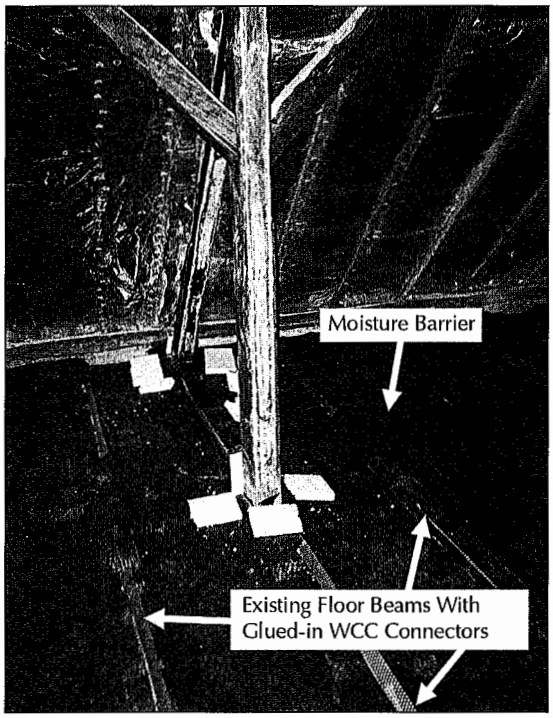

FIGURE 3. A photo of an attic floor structural upgrade of an historic home in Aschaffenburg, German, using wood-concrete technology.

turn-around times in comparison to floor replacement.

Shear Connectors

The connector plays an integral role in the performance of the composite in its function to transfer the shear between the concrete layer and the wood members. Consequently, a multitude of connectors with a wide range of effectiveness have been investigated and developed. These connectors are perhaps best categorized into four groups:

- dowel;
- sheet metal — flat or tubular;
- shear key with anchor; and,
- glued-in metal plate connectors.

These four groups of connectors are illustrated in Figure 4.

Group 1 connectors (for example: nails, screws and bolts) have the advantage of being uncomplicated to install (particularly for uneven surfaces that are typically encountered in restoration projects). However, they are generally considered to be the least rigid of all systems[12] and have been corroborated as such in comparative studies.[13,14] A study by Ahmadi and Saka investigated various configurations of high-strength nails, screws and bolts and found increases in static four-point flexural strength of 55 percent and reductions in mid-span deflections in the order of 2.3 times when compared to tests with no shear connectors.[1] Similarly, in another study a dowel-type connector in a fiber-reinforced-polymer glulam-concrete bridge girder was tested.[15] This study found that "stiffness increases of over 200 percent and strength gains of over 60 percent relative to the expected response of a noncomposite girder."[15] Currently, screw connectors inserted at ±45° are the most commonly used shear fastener in Europe.[3]

Group 2 connectors have been shown to possess higher rigidity, ductility and ultimate strength than dowel connectors.[14,16] The reason is that nails and screws typically cause wood crushing and ultimately splitting failure, whereas sheet metal connectors (rigid rings, in particular) can cause wood shear plug failure — a stronger, more rigid mode of failure. Hollow cylinders, as well as bent sheet metal (truss-plate) connectors, are also apt to provide good ductility since the common failure modes are wood embedment, metal shear and/or concrete crushing.[17,18]

Group 3 connectors — where notches (shear keys) have been cut into the wood and have been reinforced with an anchoring device such as a post-tensioned bolt or lag screw — have been shown to have similar to moderately better strength and slip resistance than Group 2 connectors.[13,14] The horizontal shear forces are transmitted through the shear key with little interlayer slip, while the dowels work in traction to resist the vertical load component. Gutkowski *et al.* reported the load-slip response to be initially linear with abrupt partial failure (presumably of the concrete) with a modest residual ductile behavior thereafter (presumably from yielding of the dowel).[19] Further tests were performed by Gutkowski *et al.* on various adhesives to study the withdrawal strength of

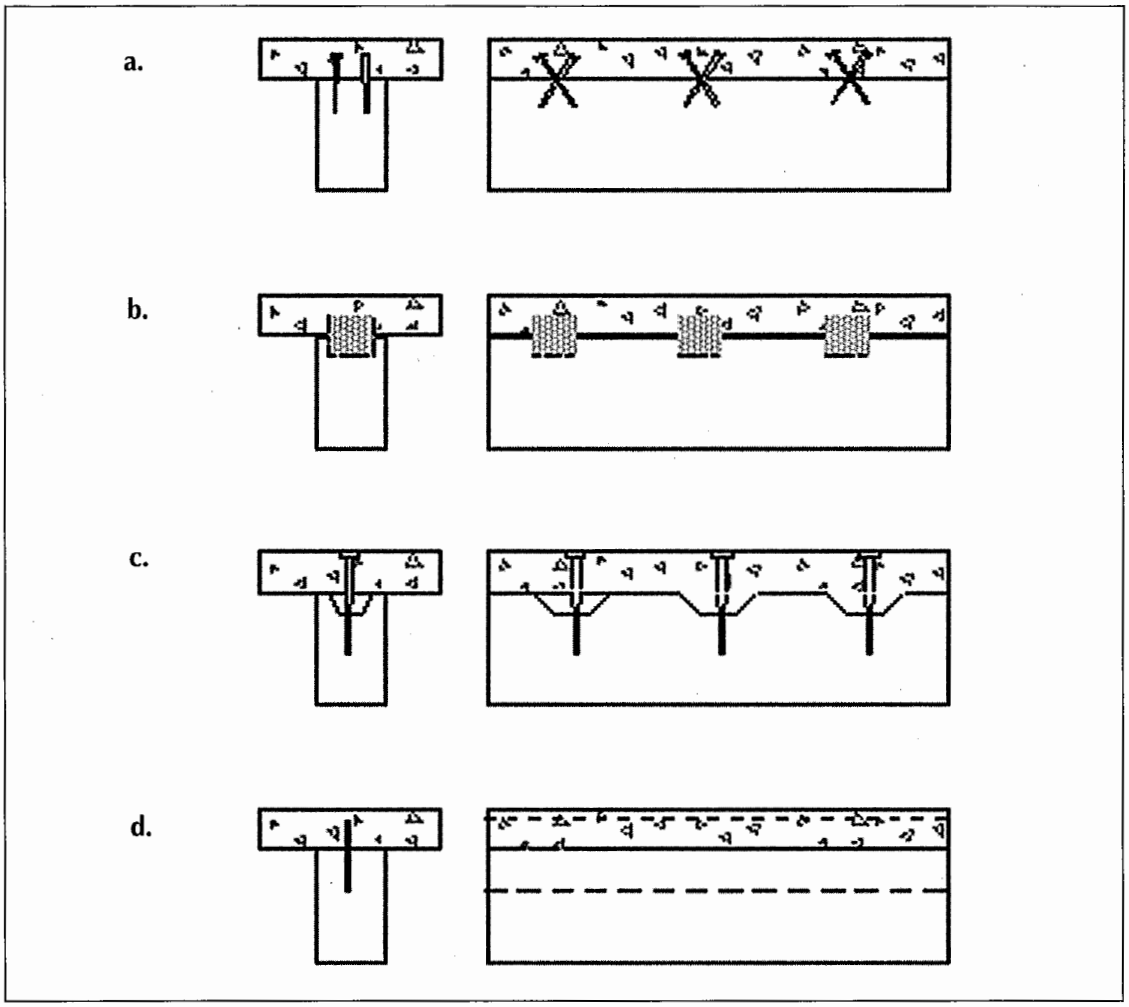

FIGURE 4. General types of sheer connectors: a) dowel; b) sheet metal; c) concrete shear key with anchor; and 4) glued-in metal plate.

the anchor.[20] Disadvantages of Group 3 connectors include the deleterious effects of reducing wood cross-section as well as a rather complicated installation process.

Group 4 connectors utilize glued-in connectors such as a continuous steel mesh, of which one half is glued into a slot in the wood while the other half is embedded into the concrete.[21,22] These systems are generally considered to possess the greatest rigidity.[12] Studies have shown that near-to-full composite action can be achieved with this type of connector.[5,22] In these studies, failure was intentionally induced in the steel plate, as opposed to the wood or concrete, in order to attain the superior qualities of steel — good predictability, high strength and stiffness, as well as high ductility. On-site construction of this system is simple and fast relative to other connector systems.

Design Methodology

Mechanical Behavior. In general, when a wood-concrete composite is subjected to positive bending, the concrete layer experiences combined bending and compressive stresses while the wood member experiences combined bending and tensile stresses. The actual linear elastic stress field is dependent on the rigidity of the shear connector (see Figure 5). The connector transfers longitudinal shear between the wood and the concrete. This shear force is in equilibrium with the internal normal force

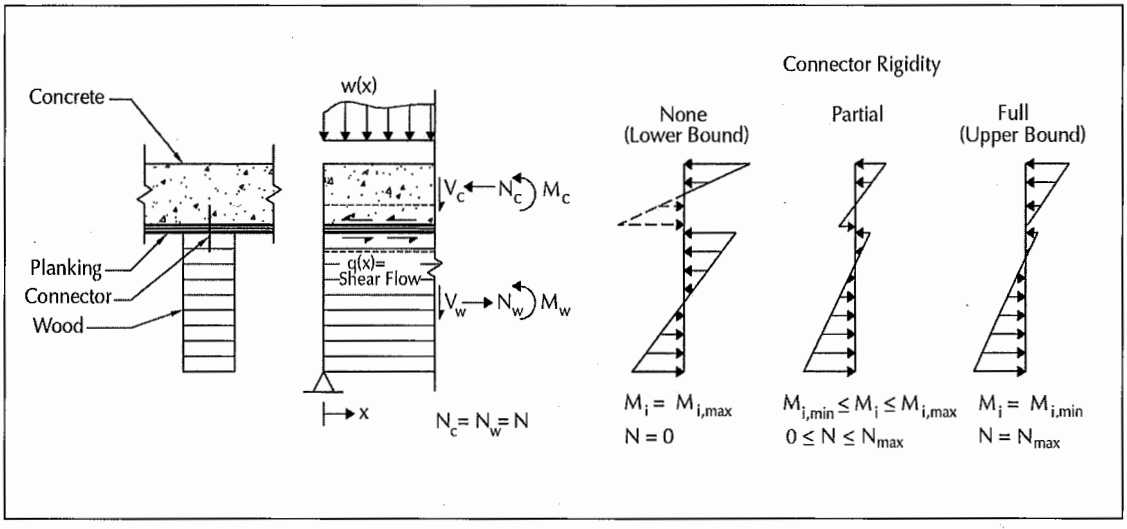

FIGURE 5. The stress field of a wood-concrete composite that is dependent on connector rigidity.

of each component. At the lower bound of rigidity, when the layers are not connected, the shear and normal forces are zero and the components are subjected only to bending. The upper bound (i.e., full composite action) occurs when the layers are rigidly connected and act as one unit with no interlayer slip. In this case, the shear force is at maximum and the components are subjected to maximum normal forces and minimum bending moments. Full composite action is convenient for design purposes because the mechanics technique of transformed sections may be applied to estimate the stress field. Also, ideally, full composite action is the desired state since it provides the most structural efficiency; however, zero slip is difficult to achieve in practice and most connectors are semi-rigid, producing some interlayer slip (i.e., partial composite action).

Design Equations. To date, no design guidelines are available in the United States for the design of WCCs with semi-rigid connectors. Some texts suggest using the method of transformed sections — as is done for steel-concrete composites, but this method is valid only for fully composite sections.[23,24] The method is non-conservative for partially composite sections.

Design equations for semi-rigid connectors *are* available, however, in the *European Standard for Timber Design,* Eurocode 5, Appendix B.[25] It is assumed in the derivation of these equations that, for linear elastic behavior, the Bernoulli theory of elementary mechanics (plane sections remain plane) does not apply to the cross-section as a whole, but is valid for each component. The stress field is the algebraic summation of internal normal stresses and bending stresses within each component (see Figure 6).

For a simply supported wood-concrete composite beam with span l, the effective bending stiffness, $(EI)_{ef}$, is calculated as:

$$(EI)_{ef} = \sum_{i=1}^{2} (E_i I_i + \gamma_i E_i A_i a_i^2) \quad \textbf{(1)}$$

where (referencing Figure 5):

subscripts $i = 1$ and $i = 2$ refer to the respective component;
E = modulus of elasticity;
I = moment of inertia;
A = cross-sectional area;
a = distance from centroid of respective component to overall neutral axis; and
γ = a dimensionless shear connection reduction factor.

The value of γ_1 ranges between 0 (no composite action) and 1 (full composite action) and is calculated as:

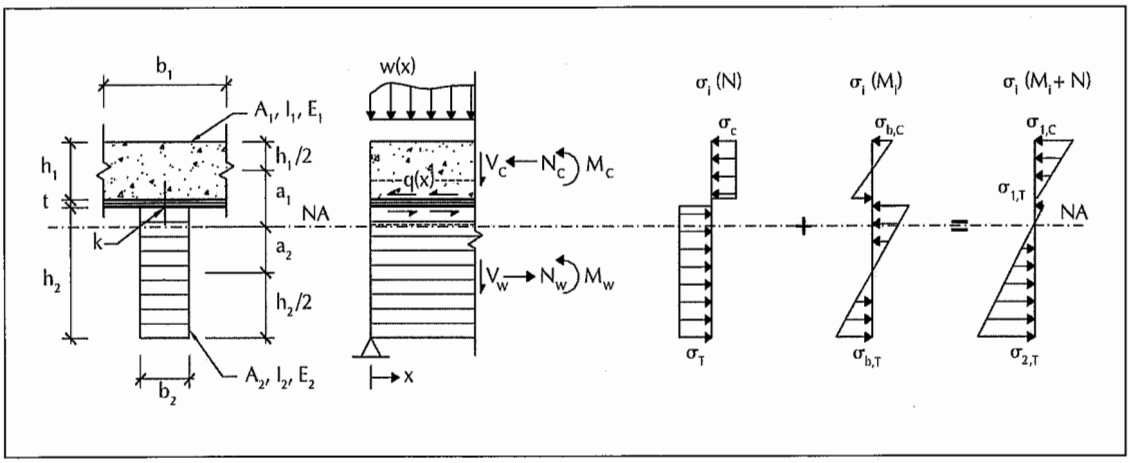

FIGURE 6. The stress field of a wood-concrete composite with semi-rigid connectors.

$$\gamma_1 = \frac{1}{1+\dfrac{\pi^2 E_1 A_1 s}{Kl^2}} \; ; \; \gamma_2 = 1.0 \qquad \textbf{(2)}$$

where:

s = spacing of the connectors;
K = slip modulus; and
l = beam span.

The slip modulus, K, is determined as the slope of the load/slip curve from an experimental test. The value of γ_2 is a constant value of 1.0.

The distance between the centroid of the wood member and the overall neutral axis, a_2, is dependent on the shear reduction factor, γ_1, and is calculated as:

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1+h_2+2t)}{2\sum_{i=1}^{2} \gamma_1 E_1 A_1} \qquad \textbf{(3)}$$

where:

h_1 = height of the concrete;
h_2 = height of the wood; and
t = height of the planking.

The distance a_1 is determined from the geometry of the composite beam and Equation 3.

The maximum compressive stress in the concrete, $\sigma_{1,C}$, occurring at the upper surface of the concrete, is calculated as:

$$\sigma_{1,C} = \sigma_C + \sigma_{b,C} = \frac{M}{(EI)_{ef}} \gamma_1 E_1 a_1 + \frac{M}{(EI)_{ef}} \frac{h_1}{2} E_1 \qquad \textbf{(4)}$$

where:

σ_C = normal compressive stress in the concrete due to the force couple in the composite section;
$\sigma_{b,C}$ = maximum compressive bending stress in the concrete due to the force couple about the concrete section; and
M = applied moment.

The maximum tensile stress in the concrete, $\sigma_{1,T}$, occurring at the lower surface of the concrete is simply the subtraction of the tensile bending stress, $\sigma_{b,T}$, from the compressive stress, σ_C, in the concrete:

$$\sigma_{1,T} = \sigma_C - \sigma_{b,T} = \frac{M}{(EI)_{ef}} \gamma_1 E_1 a_1 - \frac{M}{(EI)_{ef}} \frac{h_1}{2} E_1 \qquad \textbf{(5)}$$

The maximum tensile stress, $\sigma_{2,T}$, in the wood is calculated in a similar manner as:

$$\sigma_{2,T} = \sigma_T + \sigma_{b,T} = \frac{M}{(EI)_{ef}} E_2 a_2 + \frac{M}{(EI)_{ef}} \frac{h_2}{2} E_2 \qquad \textbf{(6)}$$

where:

σ_T = tensile stress in the wood due to the force couple in the composite section;
$\sigma_{b,T}$ = maximum tensile bending stress in the wood due to the force couple about the wood section; and
M = applied moment.

The maximum beam shear stress — which occurs at the neutral axis in the wood component — can be calculated using:

$$f_v = \frac{E_2 h^2 V}{2(EI)_{ef}} \quad (7)$$

where:

V = applied shear force; and

$h = a_2 + h_2/2$

The shear flow, q, in the connector may be computed as:

$$q = \frac{\gamma_1 E_1 A_1 a_1 V}{(EI)_{ef}} \quad (8)$$

Design & Construction Considerations

The initial design is typically performed using the full concrete height (uncracked section). If it is found that tension exists in the concrete, this assumption may have to be revised in further iterations. Consistent with other design practices, the modulus of elasticity (MOE) of both materials is usually taken as the mean value. It is possible to gain in economy by staggering the shear fasteners according to the shear stress distribution. It is also possible to use more than one row of fasteners, particularly under concentrated loads or at the supports — if wood thickness permits. The designer is advised to check code requirements or product specifications issued by a fastener manufacturer as to the permissibility of this practice.

Long-term system performance of WCCs is not, as yet, well understood, particularly for fluctuating outdoor climates. It is known that wood and concrete exhibit creep under prolonged loading. For both materials, the amount of creep-related deflection is a function of load duration as well as the applied stress level. For wood, however, its creep behavior is also influenced by moisture and temperature changes due to its hygroscopic nature. While temperature has a negligible influence under normal service conditions,[26] elevated moisture levels and particularly changes in moisture content can create deflections in the range of three to four times the instantaneous creep.[27] Since wood and concrete are adjoined in a WCC, different creep rates (concrete creeps faster than wood) may also have a detrimental effect, particularly in a range between three and seven years of service.[27] In addition to magnified deflections, the differential creep also tends to migrate internal forces from the concrete to the wood, leading to higher stresses in the wood.[12]

A common approach to estimating the long-term behavior (deflection and stresses) of a wood-concrete system is through the reduced-modulus method where the modulus of elasticity for each material is reduced to account for creep.[12] Although this simplified approach may be adequate for constant environments (such as indoors), it has been shown that seasonal climate changes, such as what would occur for a bridge structure, may lead to a more unfavorable creep behavior, which cannot accurately be estimated by the reduced-modulus method.[27] Further investigation is necessary in this area and work is currently being carried out by several researchers.[28–30]

With reference to Group 4 connectors, there are some limitations to using adhesive in structural assemblies. For example, the use of dry wood (i.e., old interior timbers or kiln-dried new material) is generally a necessity for adequate adhesive bonding. Another drawback is the uncertainty of how the glue performs under temperature fluctuations (i.e., outdoor climate or fire) or under long-term loading. For example, standard light-frame construction adhesive is often neglected in strength or deflection calculations for medium- to long-duration loads such as dead or floor live load. However, a wide array of high-strength adhesives exist that are well suited to structurally demanding applications. For example, in concrete construction, anchoring epoxies are commonly used and, in wood construction, phenol resorcinol adhesive has undergone extensive research (and more than 100 years of service in glulam) to prove its reliability for structural use. Comprehensive understanding of the adhesive's behavior is a critical component of the development of this connector system and, to that end, testing is ongoing at the University of Massachusetts, Amherst.

Ease of installation is often a decisive factor when choosing a shear connector. While Group 1 fasteners are uncomplicated to install

and do not require routing of the wood (as would Group 3 connectors), usually a large number of fasteners are necessary — meaning substantial nailing, screwing or pre-drilling with subsequent insertion of the dowel. A more labor-efficient approach is employed for a Group 4 connection system, where only three steps are needed:[22]

1. Cut a 3.5-millimeter (0.14-inch) wide and 40-millimeter (1.6-inch) deep saw kerf in the wood member.
2. Place glue into the kerf.
3. Push the steel sheet into the assembly.

Both Steps 1 and 2 are continuous processes that can be simplified greatly by power tools.

In many structures, the horizontal structural elements (floors, roof) are used to transfer wind loads to vertical load-resisting elements (for example, shear walls). When a floor acts in this manner (as a diaphragm), its in-plane stiffness is the determining factor as to how much load each shear wall will receive. If a diaphragm is rigid, interior and exterior shear walls typically experience similar loads. If the diaphragm is flexible, however, interior walls tend to receive higher loads than external walls. The addition of a monolithic concrete slab on top of the wooden beams in the WCC system increases in-plane stiffness and provides a more rigid diaphragm than a wooden floor system with decking alone. Thus, when used to renovate masonry buildings with wooden floors (such as mill buildings) the effects of the increased diaphragm rigidity on the lateral load-resisting system of the building should be considered.

The fire performance of the wood-concrete system needs to be evaluated with regard to three characteristics:

- smoke impermeability;
- thermal insulation; and,
- structural stability and strength.

In regards to smoke impermeability, the monolithic concrete slab of the WCC is very effective in preventing smoke from permeating between floors. However, as with any floor system, proper detailing must be provided at openings that penetrate the slab. Regarding thermal insulation, the thermal properties of each material must be considered separately. Typical thermal conductivity, k, for concrete is 5 to 10 Btu/ft^2/°F/in/hr and for wood 1 to 2 Btu/ft^2/°F/in/hr.[31] This difference favors the slab-type WCC system (see Figure 2), where the solid wood slab provides a highly effective thermal barrier between floors. Also, since the (usually metal) WCC-connector is typically inserted into the center of the top side of a beam, it is embedded enough into the wood and the concrete to have sufficient insulating material around it. In terms of structural stability and strength, two strategies can be pursued. For the first strategy, it is important to note that during a fire, the cross-section of a wooden member is reduced at a very consistent rate of approximately 0.6 millimeters (0.023 inches) per minute on average.[32] The undestroyed core remains fully capable of carrying load. If the wood cross-sections (for either WCC system) are increased by the necessary amounts, fire-resistance ratings of 1 hour can easily be achieved. An increase in cross-sections will likely also yield dimensions beyond the 140 millimeters (6 inches nominal) that many building codes require for a "heavy timber" designation. An alternative approach is to use other methods of fire protection (for example, spray-on fireproofing, suspended ceilings and sprinklers) in conjunction with a WCC system.

With respect to construction, several issues should be considered. The slab is typically cast in place and transverse planking acts as permanent formwork as well as the finished ceiling. To reduce the service load stresses, temporary shoring may be used to support the weight of the timber beams and wet concrete prior to curing. After the concrete cures, the shores are removed and the section acts compositely to resist all loads.

It is important to protect the wood from direct contact with wet concrete either during concrete curing or in service. Wet wood can lead to excessive deflection and, if not dried promptly, decay. Wood protection is usually assured by placing a water barrier film on top of the planking (typically one or two layers of 0.2-millimeter

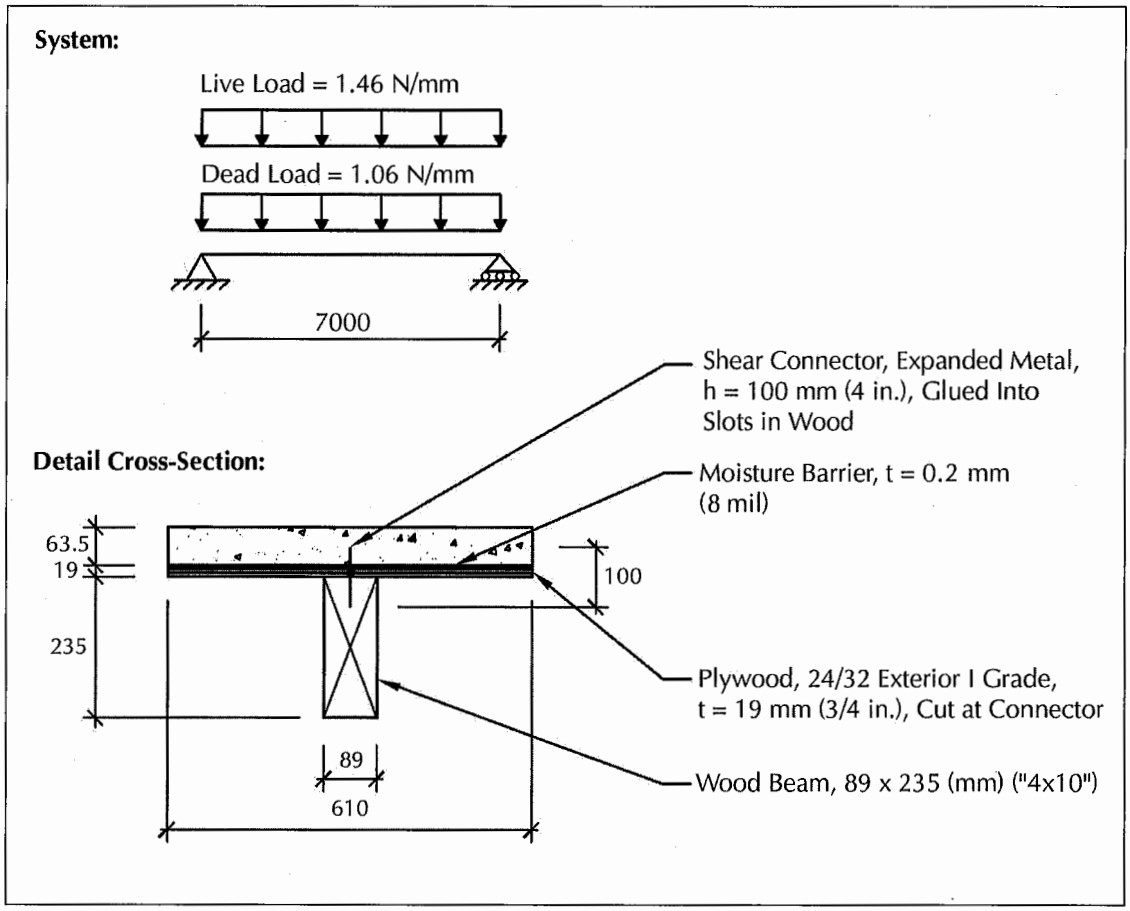

FIGURE 7. An example application of a commercial wood-concrete floor (all dimensions are in millimeters).

[0.008-inch] polyethylene [PE] foil). When Group 4 connectors are used, any cuts in the foil from cutting the saw kerf for the connector may be sealed by extra glue squeezed out from the placement of the connector.

Other construction issues for WCCS are:

- long-span applications are often designed with some camber to offset dead load deflection;
- adequate bearing surface should be provided to avoid wood crushing at the supports; and,
- a small ventilation gap should be provided around timber beams that rest in masonry pockets.

If the composite section design check indicates that the concrete slab acts fully in compression, then only minimum reinforcement (to prevent cracking) is required.

Design Example

The following design example outlines a suggested allowable stress design (ASD) procedure for a WCC floor with semi-rigid connectors. This approach is used (as opposed to a load and resistance factored design [LRFD] method) because the WCC system is relatively new and strength reduction factors, used to calibrate the LRFD method, are not available. Design of the wood component follows the 2001 National Design Specification (NDS) for Wood Construction,[33] the concrete properties derive from ACI 318-02[34] and loads are applied in accordance with the 2003 edition of the International Building Code (IBC).[35]

Figure 7 shows a typical commercial application of a wood-concrete floor spanning 7 meters (23 feet) with a 63.5-millimeter (2.5-inch) thick medium density concrete slab. The wood beams (visually graded southern pine, No. 1 dimension lumber) are spaced at 610 millimeters (24 inches) on center. They are 89 millimeters (3.5 inches) wide by 235 millimeters (9.25 inches) deep (i.e., nominal 4 by 10s). The planking is 19-millimeter (0.75-inch) thick plywood (such as a 24/32 structural panel). The dead load on the assembly is 1.06 kN/m (72.6 plf). Live load is the minimum uniformly distributed load per the IBC for offices — 1.46 kN/m (100 plf). The composite beams will be supported on temporary shoring while the concrete cures so that the composite section will act to resist all dead and live loads. If the construction were without temporary shoring, the wood beams would be required to act non-compositely (i.e., by itself) to support the weight of the wet concrete as well as any formwork and its self weight.

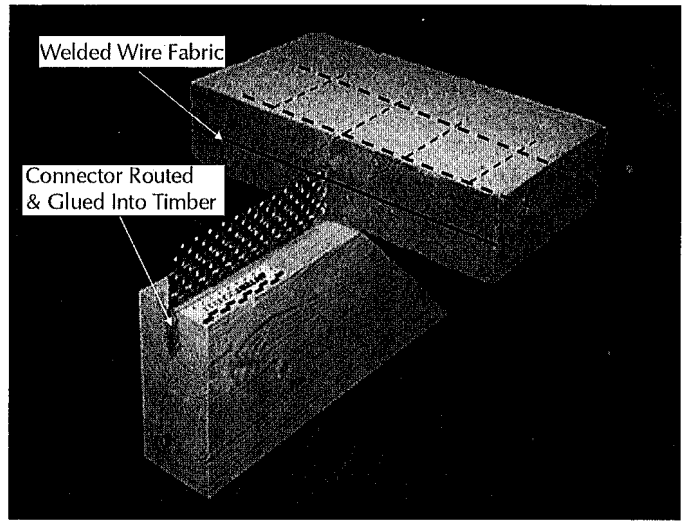

FIGURE 8. Connector detail for the example application.

Material Properties. The material properties of the medium-density concrete are: modulus of elasticity, E_1, of 23,000 MPa (3335.9 ksi); and a specified compression strength, f'_c, of 25 MPa (3625.9 psi).

The material properties of the wood (No. 1 grade southern pine) used: modulus of elasticity, E_2, of 11,700 MPa (1696.5 ksi); unadjusted parallel to grain tension strength, F_t, of 7.24 MPa (1050.1 psi); unadjusted bending strength, F_b, of 12.76 MPa (1850.7 psi); and, unadjusted shear strength, F_v, of 1.21 MPa (175.5 psi).

The connector to be used in this floor (see Figure 8) is a glued-in expanded-metal plate with a total connector slip modulus, K, of 415,460 N/mm (2.37×10^6 lb/in) and strength, Q, of 279 N per millimeter length (1593 lb per inch length) of connector.[22] Both properties are average values determined from experimental tests using a connector length of 400 millimeters (15.7 inches). The equivalent slip modulus K per millimeter length of connector (or beam) is calculated as 1039 N/mm per millimeter length (151×10^3 lb/in per inch length).

It is noted that to provide adequate protection of the glued connection against failure, a minimum cross section of 80 by 80 millimeters (3.1 by 3.1 inches) should be used for the wooden beams.[36] Doing so prevents the use of this WCC connector (without modification) in residential construction, where floor joists commonly measure 38.1 millimeters (1.5 inches) in width. Primary applications are in heavy-timber construction, upgrading of mill buildings or other commercial construction.

Composite Factors. Using Equation 2, the reduction factor, γ_1, is found to be equal to 0.85. This value is a highly favorable result in light of the fact that as γ_1 approaches 1, the system approaches full composite action. Moreover, for nails and screw type connectors, the factor is typically much less — with γ_1 between 0.1 and 0.4.[37]

Using Equation 3, and given A_1 equal to 38,735 square millimeters (60 square inches) and A_2 equal to 20,915 square millimeters (32.4 square inches), the distance between the centroid of the wood member and the overall neutral axis, a_2, is calculated to be 127.3 millimeters (5 inches). Using this result in

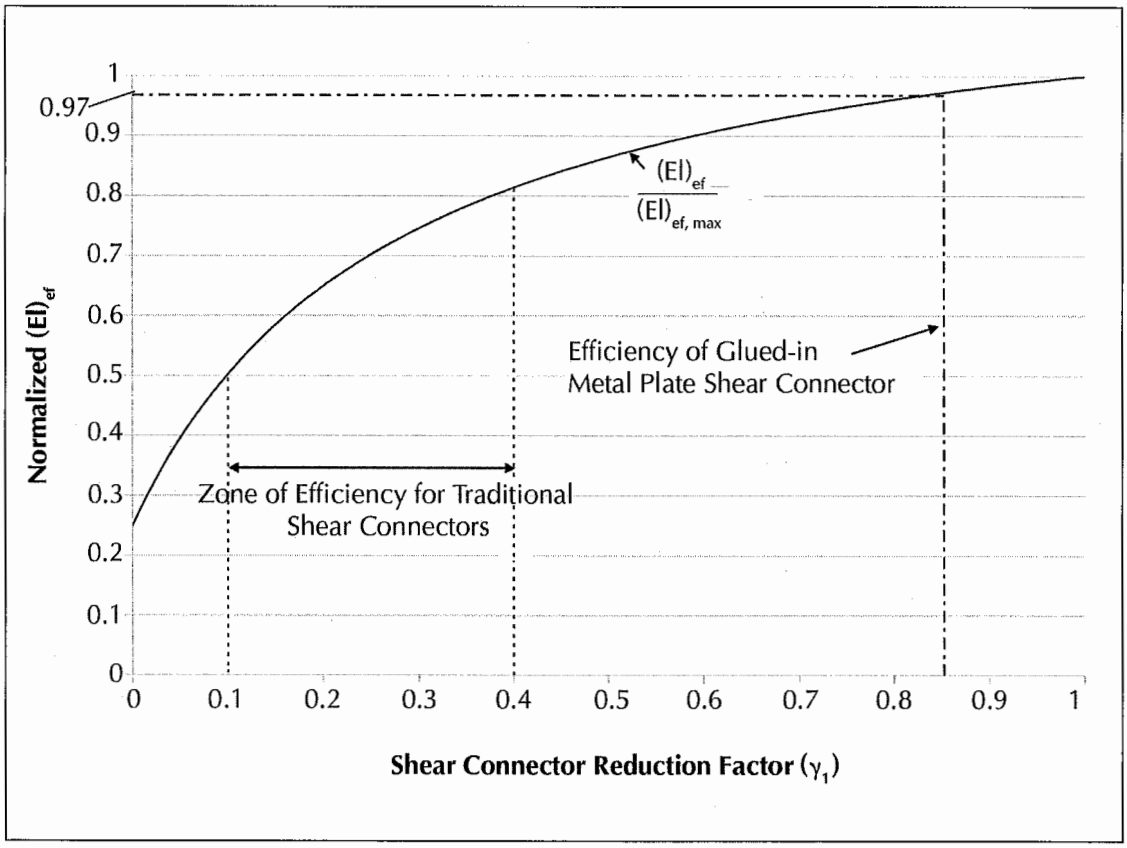

FIGURE 9. Effective bending stiffness as it varies with the shear reduction factor, γ_1. This plot shows the high efficiency of glued-in metal plate connectors in comparison with dowel connectors.

Equation 1, and given a_1 equal to 41.0 millimeters (1.6 inches), I_1 would be 1.3×10^7 mm^4 (31.2 in^4) and I_2 equal to 9.63×10^7 mm^4 (231.4 in^4) the effective bending stiffness, $(EI)_{ef}$, equal to 6.66×10^{12} N·mm^2 (2.32×10^9 lb·in^2).

Figure 9 gives a visual depiction of the efficiency of this connector for this application. The graph plots the increase of effective bending stiffness, $(EI)_{ef}$, with γ_1. For the given application, the effective bending stiffness, $(EI)_{ef}$, would be 6.66×10^{12} N·mm^2 (2.32×10^9 lb·in^2), which is 97 percent of (or 3 percent softer than) that of the most efficient wood-concrete system (i.e., when γ_1 is 1) where $(EI)_{ef}$ is 6.89×10^{12} N·mm^2 (2.40×10^9 lb·in^2).

Strength Analysis (t equal to 0, Short-Term). In terms of wood tensile failure, a linear interaction formula is employed:

$$\frac{\sigma_T}{F_t'} + \frac{\sigma_{b,T}}{F_b^*} \leq 1$$

where:

σ_T = tensile stress in the wood due to the force couple in the composite section;

$\sigma_{b,T}$ = maximum tensile bending stress in the wood due to the force couple about the wood section;

F_t' = allowable parallel-to-grain tensile stress; and

F_b^* = allowable bending tensile stress without adjustment for lateral stability.

For applied stress, the governing load combination is w equal to D plus L. Hence, the maximum applied moment, M, is equal to $w \cdot l^2$ divided by 8, or 1.55×10^7 N·mm (1.37×10^5 lb·in). Using Equation 6 and knowing that a_2 is 127.3 millimeters (5 inches) and $(EI)_{ef}$ is $6.66 \times$

10^{12} N·mm^2 (2.32×10^9 lb·in^2), then σ_T would be 3.45 MPa (500.4 psi) and $\sigma_{b,T}$ would be 3.19 MPa (462.7 psi).

In terms of strength, the allowable parallel-to-grain tensile stress, F_t', would be $F_tC_DC_MC_tC_FC_i$ where the C-factors represent adjustment factors to account for load duration (C_D), moisture (C_M), temperature (C_t), member size (C_F) and incising (C_i). Assuming a dry service condition, normal temperature range and untreated material, all adjustment factors default to 1.0. Therefore, F_t' is F_t, or 7.24 MPa (1050.1 psi).

The allowable bending tensile stress without adjustment for lateral stability, F_b^*, would be $F_tC_DC_MC_tC_FC_iC_r$ where the C-factors are as noted as above but also include a factor to account for repetitive member design (C_r). In this case, the only applicable factors are C_F equal to 1.1 and C_r equal to 1.15. Therefore, F_b^* is $F_bC_FC_r$, or 12.7 MPa·1.1·1.15, which is 16.07 MPa (1842 psi·1.1·1.15, or 2330.8 psi).

Finally, using the interaction equation:

$$\frac{\sigma_T}{F_t'}+\frac{\sigma_{b,T}}{F_b^*} = \frac{3.45}{7.24}+\frac{3.19}{16.07} = 0.68 \leq 1 \rightarrow \text{okay}$$

In terms of wood shear failure, the maximum shear stress in the wood, f_v, is checked against the allowable shear stress, F_v':

$$f_v \leq F_v'$$

For applied stress, the maximum design shear in the beam is V equal to $w \cdot l$ divided by 2, or 8.84 kN (1987.3 lb). The corresponding applied shear stress, f_v, is 0.46 MPa (66.7 psi).

For strength, the allowable shear stress, F_v', is equal to $F_vC_DC_MC_tC_i$ where the C-factors represent adjustment factors as noted above. In this case, all factors default to 1.0. Therefore, F_v' equals F_v, or 1.21 MPa (175.5 psi).

Considering the design equation (Equation 7) for shear:

$$f_v = \frac{E_2h^2V}{2(EI)_{ef}} = 0.46\ MPa \leq F_v' = 1.21\ MPa$$
$$(66.7\ psi \leq 175\ psi) \rightarrow \text{okay}$$

In terms of concrete compressive failure, the maximum compressive stress in the concrete, $\sigma_{1,C}$, is checked against the allowable concrete compressive strength, F_c:

$$\sigma_{1,C} \leq F_c$$

For applied stress, given the applied moment, M, of 1.55×10^7 N·mm (1.37×10^5 lb·in), a_2 of 127.3 millimeters (5.0 inches) and $(EI)_{ef}$ of 6.66×10^{12} N·mm^2 (2.32×10^9 lb·in^2) and using Equation 4, then $\sigma_{1,C}$ is equal to 3.56 MPa (516.3 psi).

The allowable concrete compressive strength, F_c, is assumed to be one-half the specified compressive strength, f'_c.[38]

The design check for concrete becomes:

$$\sigma_{1,C} = 3.56\ MPA \leq F_c = 12.5\ MPa$$
$$(516.3\ psi \leq 1813\ psi) \rightarrow \text{okay}$$

In terms of concrete tensile failure, the maximum tensile stress in the concrete, $\sigma_{1,T}$, is calculated using Equation 5. Given the applied moment, M, of 1.55×10^7 N·mm (1.37×10^5 lb·in), a_2 of 127.3 millimeters (5.0 inches) and $(EI)_{ef}$ of 6.66×10^{12} N·mm^2 (2.32×10^9 lb·in^2), then $\sigma_{1,T}$ is equal to 0.17 MPa (24.7 psi) in compression. Hence, the concrete slab is subjected to compression stresses exclusively and tensile failure is not a consideration.

In terms of connector shear failure, the maximum shear flow in the connector from the applied loads, q, is checked against the allowable shear capacity of the connector, Q_a:

$$q \leq Q_a$$

For applied stress, the maximum applied shear force, V, is 8.84 kN (1987.3 lb). Using Equation 8, the maximum shear flow experienced by the connector, q equals 41.3 N/mm (235.8 lb/in).

The allowable connector strength, Q_a, is assumed to be one-third the specified shear strength, Q.

The design check for the connector is thus:

$$q = 41.3\ N/mm \leq Q_a = 93.0\ N/mm$$
$$(235.8\ lb/in \leq 531.0\ lb/in) \rightarrow \text{okay}$$

Serviceability Analysis (t equal to 0, Short-Term). Because the floor is temporarily shored, the composite section is expected to carry both

the dead load and the live load. Consequently, the effective stiffness of the composite section — $(EI)_{ef}$ equals 6.66×10^{12} N·mm^2 (2.32×10^9 lb·in^2) — is used to calculate live load deflection as well as dead load deflection.

The dead load deflection, Δ_{DL}, is calculated thus:

$$\Delta_{DL} = \frac{5wL^4}{384(EI)_{ef}} = \frac{5(1.06)(7000)^4}{384(6.66)(10)^{12}}$$

$$= 5.0\ mm \left(\frac{5(6.05)(275.6)^4}{384(2.32)(10)^9} \right) = 0.20\ in$$

If desired, the composite beam can be manufactured with a camber to offset this deflection.

The live load deflection, Δ_{LL}, is calculated thus:

$$\Delta_{LL} = \frac{5wL^4}{384(EI)_{ef}} = \frac{5(1.46)(7000)^4}{384(6.66)(10)^{12}}$$

$$= 6.8\ mm \left(\frac{5(8.34)(275.6)^4}{384(2.32)(10)^9} \right) = 0.27\ in$$

One of the primary advantages of a wood-concrete system is the superior stiffness afforded by composite action. In this example, the live load deflection is relatively small, providing a serviceability of $L/1029$. In comparison, code guidelines for deflection limits typically suggest that for walking comfort (or to avoid plaster cracking) of commercial beams, the live load limit can be as much as $L/360$.

Strength & Serviceability Analyses (t equals ∞, Long-Term). No consensus among researchers has been reached for estimating the long-term performance of WCC beams. As noted above, one common approach has been to approximate the creep behavior by reducing the modulus of elasticity for each material. For steel-concrete construction, the ACI-ASCE Joint Committee recommends using $E_c/2$ as the concrete modulus of elasticity instead of E_c when calculating sustained load creep deflection.[39] The AASHTO Bridge Design Specification, Section 10.38.1.4, suggests using $E_c/3$.[40] The European Code recommends using creep factors developed through load duration studies to reduce the moduli of the respective materials.[25] In absence of an established approach, the European method is adopted for this example (see Table 1).

Further Analysis. In addition to the aforementioned design checks, the analyses on the following items should also be performed (as appropriate):

- wood bearing at the supports;
- planking between beams; and,
- concrete plate between beams.

Lateral stability of the wood beams is provided through the concrete slab — therefore, a design check on that item should not be necessary.

Conclusion

WCC floor systems make use of a well established mechanics principle — that of composite action. Composite action is developed when two materials are integrally connected and deflect as a single unit, which results in a floor that exhibits greater strength and stiffness than a floor where the supporting members act independently. Direct advantages of composite design include increased floor stiffness, increased span length, shallower floor beams and/or reduction in weight of the floor system. Advantages also stem from the integration of wood with concrete — adding concrete to a timber floor will naturally improve sound and vibration performance as well as fire resistance and durability. This system also provides the aesthetics of a finished wood ceiling directly after the completion of the primary structural members. Furthermore, the inclusion and promotion of wood into a structural system generally delivers environmental benefits.

A mechanical shear connector is used to resist the horizontal shear that develops during bending. Several options exist and different connector systems provide different levels of composite action. While it is acceptable to use transformed-section analysis for steel-concrete composites, which can acquire full composite action, this procedure is not appropriate for WCCs, which typically exhibit only partial composite action. There is a need in the United States for an adequate design method that can account for semi-rigid shear connec-

TABLE 1.
Long-Term Strength & Serviceability Analyses

Given:

ϕ_1 = creep factor for concrete for permanent loads = 2.25

$E_{1(reduced)} = E_1/(1+\phi_1) = E_1/(1+2.25) = 23{,}000/3.25 = 7077$ MPa (3335.9/3.25 = 1026.4 ksi)

ϕ_2 = creep factor for timber for permanent loads = 0.25

$E_{2(reduced)} = E_2/(1+\phi_2) = E_2/(1+0.25)\ 11{,}700/1.25 = 9360$ MPa (1696.9/1.25 = 1357.5 ksi)

$(EI)_{ef(reduced)} = 4.16 \times 10^{12}\ N{\cdot}mm^2\ (1.45 \times 10^9\ lb{\cdot}in^2)$

Strength

(Calculated in manner analogous to the aforementioned design checks.)

Wood:

Bending & Tension:

$$\frac{\sigma_T}{F_t'} + \frac{\sigma_{b,T}}{F_b^*} = \frac{3.34}{7.24} + \frac{4.09}{16.07} = 0.72 \leq 1 \quad \left(\frac{484.4}{1050} + \frac{593.2}{2330.8} = 0.72 \leq 1\right) \rightarrow okay$$

Shear:

$f_v = 0.46$ MPa $\leq f_v' = 1.21$ MPa (66.7 psi $\leq$ 175.5 Psi) → okay

Concrete:

Compression:

$\sigma_{1,C} = 2.64$ MPa $\leq F_c = 12.5$ MPa (382.9 psi $\leq$ 1813 psi) → okay

Tension:

$\sigma_{1,T} = 0.97$ MPa (140.7 psi) in compression → okay

Fastener:

Shear:

$q = 40.0$ N/mm $\leq Q_a = 93.0$ N/mm (228.4 lb/in $\leq$ 531.0 lb/in) → okay

Deflection

For sustained load deflection, only the dead load need be considered:

$$\Delta_{DL} = \frac{5wL^4}{384(EI)_{ef}} = \frac{5(1.06)(7000)^4}{384(4.16)(10)^{12}} = 8.0\ mm \left(\frac{5(6.05)(275.6)^4}{384(1.45)(10)^9}\right) = 0.31\ in$$

This creep deflection represents a serviceability of $L/875$ and is acceptable.

tors for the design of WCCs. Suggested design equations from the European Standard for Timber Design, Eurocode 5, Annex B have been presented herein and have been used to demonstrate a sample design analysis of a typical commercial WCC floor structure.[25]

Research is currently underway in many places throughout the world to better understand aspects of WCCs. A major focus has been placed on studying and modeling rheologic (time-dependent) behavior. Not knowing the long-term performance of WCCs, particularly outdoors under fluctuating moisture conditions, is a barrier to widespread use of these systems. Other issues specific to the shear connector, such as adhesive performance or connector dimensions, are also being examined by individual researchers.

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The Effective Use of Commercial Computer Software for the Structural Design of Buildings

While past developments in structural software provided modest gains on the design process, new developments may require engineers to redefine their roles and responsibilities.

JAMES C. PARKER, PEDRO J. SIFRE & MICHAEL J. BOLDUC

To state the obvious, historical records show that there are many fine examples of structural engineering achieved prior to the use of computers in engineering. A couple of these notable structures are the Brooklyn Bridge (in 1883) and the Empire State Building (in 1931). Thousands of more humble yet equally successful examples exist. These works are a result of sound structural design without the use of commercial computer software. Before computers, engineers used empirical design rules, approximate analysis methods, rules-of-thumb and their engineering experience and judgment. Many of these structures may be conceptually very simple compared to modern-day counterparts such as the Boston Convention & Exhibition Center or the Ray and Maria Stata Center at the Massachusetts Institute of Technology. Perhaps computer analysis of some historic structures can demonstrate a degree of over-design or a flawed understanding of stresses and strains. Yet structural design achievements before the use of computers are still very impressive and many would be impressive if engineered today with modern computer tools.

Whereas engineers in the past (before computers) needed practical means and methods to arrive at a design, today's engineers need similar practical means and methods to check designs derived from computer software and

to better understand the structure. Some of the knowledge base used in the nineteenth and early twentieth centuries is preserved in volumes like the Kidder-Parker *Architects' and Builders' Handbook.*[1] These resources are full of interesting engineering approaches and empirical design. Although, today, engineers are less likely to need to design an unreinforced masonry arch or other archaic systems, there is a need to know the reasonable limits and capabilities of modern structural systems.

Given enough time and money, engineers today should be able to achieve excellent structural designs without computers for the vast majority of building structures. However, today's market does not give the structural engineer that luxury. It is safe to say that computers and state-of-the-art design software is a must in the modern structural engineer's office. Not because the complexities of the structure demand it (few do), but because the market demands the productivity that such tools make possible. As code-required load and design rules get more and more complicated, engineers will need to increasingly rely on computers for productivity gains. However, many should be able to continue to perform adequate designs using approximate methods without computers. Most engineers have a sense of pride about this ability, if they have it. Whether young engineers today have it or will develop it depends on the effectiveness of training and on the effective use of commercial design software. The amazing productivity gains that current and future design software provide must become a means to free the engineer from the drudgery of code checking, numerical accounting and the anxieties of mathematical errors, and provide the engineer with a greater, not lesser, understanding of the structure and its performance.

As the engineering and construction industries continue to develop new materials and new and creative ways to use them, engineers will need to develop new rules-of-thumb, approximate methods and/or empirical designs for these materials and systems. In the past, engineers needed simple, practical methods because they did not have computers or an easily accessible wealth of information. Nowadays, however, engineers need simple, practical methods to deal with the computers and an overwhelming amount of information.

It is also important to note that the impact of the personal computer (PC) on the design process goes far beyond that of engineering analysis and design software. Word processing, spreadsheets, office networks, data from the Internet and email have increased productivity and changed the way engineers work just as these technologies revolutionized the way almost all professional services do business.

Historical Perspective

The Design Process Before Computers Were Common. Although analysis programs were available for the structural engineer to use on a mainframe computer in the 1960s, computer technology did not have a major impact on the majority of engineering offices until the advent of the PC. Before the PC, the engineer performed analysis and design "by hand," and the engineer and the architect hand-drafted the contract drawings. Information was exchanged in face-to-face meetings, telephone calls and via sketches. Proximity of the engineer to the architect was important for the speed of design production. But even with engineers working in the same office with architects (as is done in architectural-engineering firms), the pace of design production could not match that of today. Changes took at least the time to re-draft the documents and even longer for changes that required reanalysis by the engineer. The amount of work behind each change led to a more deliberative design process. These time and labor demands did not decrease (and even increased) for those projects using early applications of the mainframe computer for which the engineer had to purchase precious computer time.

For a project's schematic design, the structural engineer's deliverable was usually limited to the selection of structural materials and systems. The structural engineer sketched a typical bay of framing and worked closely with the architect to identify conditions where the typical bay of framing required adaptation to fit the architecture. The engineer used approximate techniques and/or judgment to

identify important structural parameters for the architect (such as the location of braced bays, shear walls or moment frames; depths of floor framing; and load criteria). The structural engineer would start drawing production during the design development phase of the project. By that time the architect had provided the engineer with plans, building elevations and some preliminary wall sections. The engineer would periodically get drawings from the architect throughout the design development and construction document phases as the architect added information. The engineer continued to lay out the framing modifying the "typical bay" to match the architecture and performed analysis and/or empirical design to prove out the framing. It was not uncommon to have a fair degree of "over-design" in the typical conditions that allowed the engineer to apply the same designs to other variants without re-analysis.

Young designers today may not appreciate the process and time involved in the design of structural members without the use of computer software. For a building with a structural frame, three major parts comprise the design process: gravity design of columns, gravity design of floor framing and the design of the lateral-load-resisting system. A typical "design-by-hand" process could be summarized as:

Columns — After locating columns in collaboration with the architect, the structural designer calculates column gravity loads for each column. Although not necessarily a precise method of computing loads, tributary areas are calculated at each floor for each column to determine gravity axial loads in the columns. The loads are tabulated for each column at each floor for each load type — dead, live and roof live. The cumulative load in each column is modified as allowed by building code live load reduction provisions. Gravity forces from the floor framing analysis, as well as forces from the lateral load analysis, are combined together in accordance with the particular design code. Member selection is generally arrived at by selecting a trial member size and then calculating all necessary design checks. Alternate member sizes are selected until the designer is satisfied with the results of the design checks.

Floor Framing — For each floor member, the designer calculates tributary gravity loads. The designer then calculates shears and moments for each member. Forces from the lateral load analysis are added if the member forms part of the lateral-load-resisting system. Again, the forces are combined in accordance with the design code, and a trial member's strength and deflections are checked for code conformance.

Lateral-Load-Resisting System — The process includes calculating and distributing wind and seismic forces to the lateral-load-resisting frames. Of course, this process is iterative since the member sizes affect the relative stiffnesses of the frames and, thus, the distribution of forces among them. Member sizes must generate frames with sufficient lateral stiffness as well as conform to code strength checks when combined with gravity loads.

Consider the time needed for this process for each member in a large-scale building. As described above, the process requires iterations of calculations. Now, consider the additional iterations when the architecture and/or owner requirements are changing and developing during the design.

Before computers, the experience and aptitude of the structural designer had a tremendous impact on the time and quality of design. A good designer knew the design codes so well that many load combinations and design checks that would not affect the structure could be eliminated. Experience also came into play in evaluating the impact of a proposed change by the architect, and, if necessary, into defining the scope of any re-analysis and rechecking of code provisions for the areas affected by the change. For most projects, the degree of structural optimization relied on the experience and judgment of the designer.

Finally, consider the computational efforts needed for the above process to show compliance with modern codes. Earlier codes had simpler methods for developing loads, fewer

TABLE 1.
Wind Loads From the 1970 Boston Building Code
(for Minimum Wind Pressures for Height & Area Locations)

Height (ft)	P (lbs/ft^2)		
	A	B	C
0 to < 25	20	20	20
25 to < 50	25	25	20
50 to < 100	30	25	20
100 to < 150	35	30	20
150 to < 200	45	30	20
200 to < 300	45	35	25
300 to < 400	45	40	30
400 to < 500	55	45	35
500 to < 600	55	50	40
600 to < 700	55	55	45
700 to < 800	65	60	50
800 to < 900	65	65	55
900 to < 1000	65	65	60
1000+	In accordance with sound engineering principles approved by the building offical		

Note: From Ref. 2.

load combinations and, often, much simpler member stress checks. For example, comparisons between the 1970 Boston Building Code (BBC) and the provisions of the 2003 International Building Code (IBC), including the American Society of Civil Engineers — Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), illustrate the changes in the procedures and changes in what calculations would be necessary.[2,3] These comparisons lend support to the argument that today's computer-aided design software is not only a means of achieving productivity, but is also a practical necessity to avoid computational errors given the context of structural design today that includes complex codes, complex designs and fast-track construction.

Wind Loads — BBC 1970. Minimum wind pressures for a building are prescribed in a simple table. Based on building location and height, the design pressure (distributed two-thirds windward and one-third leeward) for vertical building surfaces is explicitly listed. The only wind load combination accounts for wind acting simultaneously on two perpendicular walls. In this case, a percentage (70 percent) of the design pressure shall be applied to each wall, with no further analysis required (see Table 1).

Wind Loads — IBC 2003. The IBC 2003 references ASCE 7-02 for wind load provisions. Calculations of design wind loads under these elaborate provisions include factors for basic wind speed (based on a 3-second gust), wind directionality, building importance, exposure, topography, gust effects, building enclosure, and internal and external pressure coefficients. The procedure to determine the code-prescribed wind pressure includes numerous tables, nomographs, figures and formulas. There are also multiple load cases that must be included to account for various wind load directions and load distributions. The code does provide simplified wind load provisions for regular buildings that are less than 60 feet tall, but still requires considering multiple fac-

$$p = qGC_p - q_i(GC_{pi})\ (\text{lb/ft}^2)(\text{N/m}^2) \qquad \text{(Eq. 6-17)}$$

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 I\ (\text{lb/ft}^2)$$

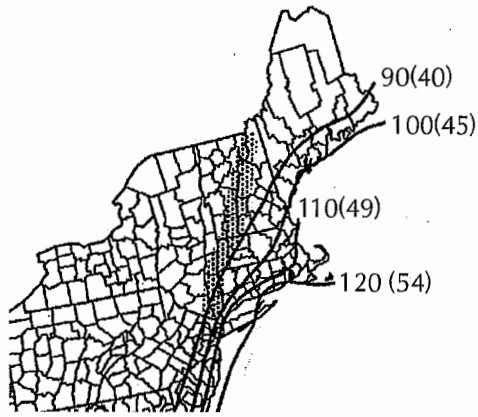
$$K_{zt} = (1 + K_1 K_2 K_3)^2 \quad K_2 = \left(1 - \frac{|x|}{\mu L_h}\right) \quad K_3 = e^{-\gamma z/L_h}$$

$$l_{\bar{z}} = c(33/\bar{z})^{1/6}$$

$$G = 0.925\left(\frac{(1 + 1.7 g_Q I_{\bar{z}} Q)}{1 + 1.7 g_v I_{\bar{z}}}\right) \quad Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\varepsilon}}$$

Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All Values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All Values	-0.7	q_h



Enclosure Classification	GC_{pi}
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

From Ref. 3

FIGURE 1. ASCE 7-02 wind load formula examples.

tors and load conditions that were not considered in previous codes (see Figure 1).

Seismic Loads — BBC 1970. Seismic loads reference the 1967 Uniform Building Code for earthquake loads and prescribe the use of Zone 2. The base shear is calculated using the formula, $V = ZKCW$. The factors (Z, K and C) determine the base shear as a percentage of the overall building mass (W) by accounting for the zone location (Z), the lateral force resisting system (K) and the period of the building (C). A simplified equivalent lateral force procedure distributes the base shear to each level of the building using the mass of each level. The code also includes a provision for horizontal torsional moments due to offsets between the center of mass and the center of rigidity.

Seismic Loads — IBC 2003. The IBC 2003 references ASCE 7-02 for seismic load provisions. The seismic loads are based on the maximum considered earthquake ground motion accelerations and response spectra. The procedure accounts for site classification, short and long periods, seismic design category, type of lateral force resisting system and structural irregularities. The equivalent lateral force procedure is still used to determine the total base shear on the building and then distribute it vertically to each floor based on mass. The code also has numerous specifications for detailing and design requirements for building components and behavior such as drift, *P*-Delta effects, soil-structure interactions and accidental torsion due to offsets between the center of mass and the center of rigidity.

Load Combinations — BBC 1970. The code is based on working stress design and requires that the engineer consider the combined effects of lateral and vertical loads. The main gravity loads including dead, live and snow loads are considered as cumulative. There is an allowance to reduce live loads for members with large tributary areas such as columns and some girders. Lateral loads such as wind and seismic loads are not required to be considered simultaneously, but they are additive to the gravity loads. Since the lateral loads are transient, the code permits a one-third

increase in the allowable working stress of the structural materials. The load combinations are, therefore, simple enough to be done by hand. Using engineering judgment, the design engineer can determine the governing load combination in each direction and limit the load combinations to just a few cases including gravity alone, and combined gravity and maximum lateral load in each direction.

Load Combinations — IBC 2003. In modern codes, the design engineer can choose to design the structural members using the allowable stress design (ASD) method or the load and resistance factor design (LRFD) method. However, new codes require considering a greater number of combinations of various gravity and lateral loads. Load combinations for both ASD and LRFD methods include a number of gravity-only cases with different factors placed on each load case. When lateral loads are combined with the gravity loads, there can be hundreds of combinations due to various load factors and load directions. Generating all the load combinations required by code, or determining the controlling case for each member by inspection is nearly impossible without the use of a computer.

Empirical Design & Approximate Analysis. Empirical design was embedded into the first American building laws at the turn of the century and is still a part of the current American Concrete Institute and The Masonry Society (ACI-TMS) masonry code today. The New York Building Law of 1882 and the Boston Building Law of 1892 both provided a schedule for determining masonry wall thicknesses based on a system of building categories and on the building's height. These provisions, along with many other empirical building design rules, can be found in the *Pocketbook* by Kidder.[4] The *Pocketbook*, in its many editions, was the authority on building construction in America from 1885 to the early 1940s.[5]

There was a great building era in America from about 1885 to 1935.[5] During this period wrought iron and steel emerged as the primary building materials. Trusses, especially steel trusses in the latter half of the period, were used in every building possible. The method of analysis most designers used for trusses consisted of graphical solutions assuming pinned connections of members. The latter half of this period also saw the proliferation of high-rise buildings that spawned explicit analysis for the lateral bracing of buildings. Designers of the period developed practical analysis methods for lateral loads for braced frames, portal frames and rigid frames. Harry Schneider illustrated these methods in his 1930 publication, *Practical Wind Bracing*.[6] The portal method and the cantilever methods are the two most well known methods, and were used predominantly prior to the proliferation of computers. Accounting for variations on, and hybrids of, these two methods, Schneider identified four methods and summarized them as follows:[6]

> "*The Cantilever Method* — The building acts as a cantilever, fixed at its base, and free to bend in a horizontal direction. The direct stresses in the columns are proportional to their distances from the center of gravity of the bent. The point of inflection of the beams is at mid-span. The vertical shear in the beams, which corresponds to the horizontal shear in an ordinary beam, increases from the outside of the bent to the center of gravity, but the increase is not uniform since the web of the cantilever is cut out between the columns.
>
> "*The Portal Method* — The [axial] stress is taken by the end columns only, the direct stresses in the center columns being zero. At any horizontal plane the wind load is divided equally among all columns. The point of inflection of the beams is not always at mid-span but must be calculated from the beam moment, which is not always the same at both ends of the beam.
>
> "*The Continuous Portal Method* — The [axial] stress in the columns is proportional to their distances from the center of gravity of the bent, the same as in cantilever method. At any horizontal plane the total wind is divided equally among all the columns, [just as] in the portal method. [As in the portal method,] the point of inflection of the beams is not always at mid-span.
>
> "*The Smith Method* — The [axial] stress is taken by the end columns only, the same as in the portal method. At any horizontal

	A	23′-4″	B		14′-0″	C		22′-8″	D		18′-0″	E
Column Moments	63		-			-			-			44
Stiffness Ratio	1/3		1/4	1/4		1/4	1/4		1/4	1/4		1/3
FEM Dead Load	-		- 91	- 37		- 37	- 70		- 70	- 59		-
FEM Total Load	- 172	+ 99	- 172	- 78	+ 73	- 78	- 147	+ 85	- 147	- 126	+ 63	- 126
	- 17	+ 11	- 29	- 1	+ 1	+ 2	- 11	+ 7	- 14	- 21	+ 13	- 7
	- 189	+ 18	- 201	- 79	- 1	- 76	- 158	+ 9	-161	- 147	+ 5	- 133
	+ 63	-	+ 30	- 30	-	- 21	+ 21	-	+ 4	- 4	-	+ 44
Max. Beam Moments	- 126	+ 128	- 171	- 109	+ 73	- 97	- 137	+ 101	- 157	- 151	+ 81	- 89
Column Moments	63		-			-			-			45

Example: (17/2)(1+1/3) = 11, (29/2)(1+1/4) = 18

From Ref. 8

FIGURE 2. Two-cycle moment distribution method.

plane, the horizontal column shears in the center columns are equal to each other, and twice as much as either end column. The point of inflection of the beams is always at the mid-span."

Moment distribution, introduced by Hardy Cross in 1929 (and eventually refined in a 1932 ASCE Proceedings paper), provided a method for determining gravity moments in a rigid frame.[7] Designers found sufficient accuracy for ordinary building frames using what became known as the two-cycle method of moment distribution (see Figure 2).[8] (Many current designers who know of this method know of it through the PCA publication *Continuity in Concrete Building Frames*.[8])

These approximate methods provide estimates of forces in the frames. In 1976, F. Cheong-Siat-Moy published a paper in the *AISC Engineering Journal* that provided a method for estimating the frame stiffness.[9] The paper uses the story drift index, which is the ratio of the relative sway of a story to its height, to designate the stiffness of a structure on a story-by-story basis. The paper proposes that the stiffness of a given story can be represented as the sum of the stiffnesses of its subassemblies, which consist of one column and the girders framing into that column (see Figure 3). The paper also provides guidelines to help increase the girders and/or the columns to meet particular stiffness requirements. By determining the frame stiffness based on the subassemblies the overall stiffness can be increased more efficiently than by increasing the girders only (which was traditionally the preferred method).

Other examples of approximate design methods are from the concrete industry as well and are still frequently used today. ACI 318 still includes provisions for estimating moments and shears in slabs and beams.[10] ACI provides coefficients for uniformly loaded beams and one-way slabs, and provides the direct design method coefficients for two-way flat slabs.

Other examples of empirical approximations include:

- Engineers traditionally estimated multistory building periods in seconds as equal to one-tenth the number of stories. (This estimate found its way into seismic building codes and appeared as recently as the 1985 edition of the Uniform Building Code.)
- Required reinforced gravity concrete column gross cross-sectional area in square inches equals the factored axial load in kips divided by half of the concrete compressive strength in ksi.
- To determine the amount of tension reinforcement in a flexural member, ACI design guides promote the expression $A_s = M_u/4d$ as a simpler alternative to the rig-

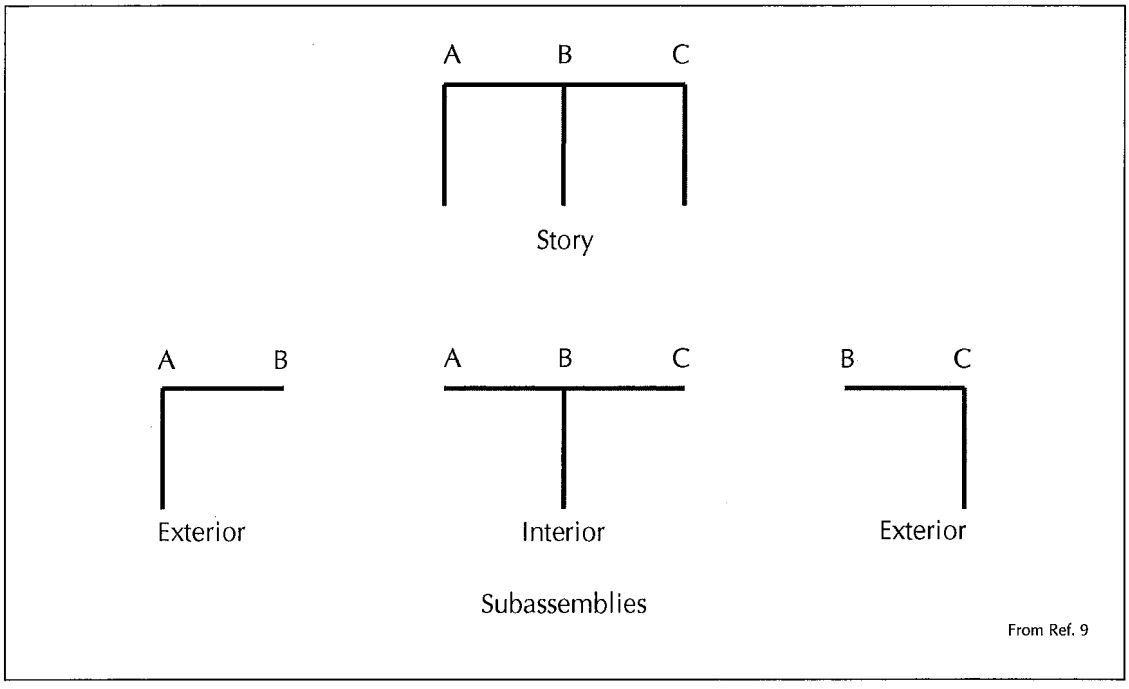


FIGURE 3. Frame stiffness subassemblies.

orous expression derived from Whitney stress block theory.

- Required steel beam depth in inches equals a certain fraction of the span in feet (0.5 to 0.7) depending on the grade of steel and on whether the section is composite or not.
- At the time when steel predominantly had a 36 ksi yield strength and ASD was the prevailing design approach, the required section modulus in cubic inches of a continuously braced beam was the moment in kip-ft divided by two.

Simplification & Common Simplifying Assumptions. It was common for designers to make many assumptions for ordinary buildings that simplified analysis and design. The designer used experience and judgment to spot the situations when an assumption would be too inaccurate. Some common assumptions included:

- Column gravity axial loads determined by tributary area were sufficiently accurate even if the columns were part of a rigid frame.
- Lateral loads were distributed to lateral-load-resisting elements assuming a completely rigid or a completely flexible diaphragm.
- Three-dimensional frame buildings could be analyzed as a series of two-dimensional frames parallel to each of the two main orthogonal axes of the building.
- AISC permitted what they called a Type 2 frame connection, where the designer sized girders or beams as simple spans for gravity loads and designed web connections for the gravity shears. Angles on the top and bottom flanges connected to the column constituted moment connections designed only for wind moments.
- Lateral loads were neglected entirely for one- and two-story buildings built with concrete frames or with steel frames with masonry partitions and exterior walls.
- Thermal stresses for ordinary buildings were negligible, especially if designers followed commonly accepted rules-of-thumb for acceptable building plan dimensions without expansion joints.
- Slabs, terracotta tied arches, planks and

other floor systems were assumed to span in only one direction.

- All truss members were typically assumed to have pin-jointed member ends regardless of the degree of continuity of the chords or of the stiffness of the gusset plate connections.
- Shear deformations of flexural members were typically ignored in deflection calculations.

The Impact of Computers. Structural analysis computer programs were in use, at least in academia, since the late 1960s. Through the 1970s and early 1980s, computer use by practitioners was generally limited to the analysis of large and/or complex structures. Designers would develop a design using approximate analysis methods and preliminary design techniques. This preliminary structure was then analyzed in detail with the computer to obtain member end forces and deformations. The "computer" was generally a large-scale mainframe computer that the designer bought time on from an off-site source. A file of "input" described the geometry, member properties, material properties, boundary conditions and loads of the model. It was routine to check input files line by line before the computer "run" since the amount of the run time could be significant and the firm was usually buying the computer time at a premium. After receiving the "output" file, the designer would check the results for verification. Verification included checking the reactions and external forces for statics, interpreting the deformed shape and comparing results with the preliminary and approximate analysis. The designer then performed design checks on the members. If the designer deemed it necessary to change members' sizes, the designer's understanding of structural analysis determined if another computer run was necessary. Generally speaking, for the building designer, the computer programs of this era were much less a design tool than an analysis tool. Compared to today's software, they were tedious and awkward to use, and offered little in the way of graphical results. Although young, inexperienced engineers may have helped the designer with checking input and output, it was necessary that the computer analysis be managed by an experienced designer or analyst to avoid the colossal waste of computer time and money.

During the 1980s and into the early 1990s, scaled-down versions of these mainframe analysis programs became available for use on the PC. Early PCs, by today's standards, had extreme limitations of memory and processor speed. However, engineering firms found them very cost effective, especially considering their additional use for word processing and spreadsheets. The analysis programs were still awkward to use since they still required input and output files. In addition, the early PC operating system, MS-DOS, was text-based. But computer time was now "free" (for the cost of the PC, software and training) and the input files were often text files that the designer could modify with text editors on the computer. Some software programs permitted the graphical display of information (such as displaced shapes and mode shapes).

By the late 1980s and early 1990s, the use of the computer for analysis was no longer limited to the most complex structures. PCs were getting faster, were less expensive and had more memory and computing power. There were still plenty of firms doing ordinary and small buildings without computer analysis, but firms that were using PCs for analysis were now using them for most structures as a productivity tool. Many firms were inventing ways to post-process the output files for convenient use with spreadsheets that did the design checks. With a PC on the desk of every designer, the role of a senior, experienced designer overseeing the use of computer analysis diminished in many firms since the "penalty" for computer model mistakes by inexperienced designers no longer amounted to the significant sums of money that were required by mainframe computer runs.

Through the 1990s, PCs continued to get faster with huge leaps in the amount of available memory, while also becoming less expensive. With the introduction of the graphical user interface provided by Microsoft Windows, designers now had powerful graphic tools to generate, display and check the computer models. At the same time, there

were rapid advancements in computer-aided design and drafting software. By 2000, with the Internet going mainstream, email and intra-office networks, there was a PC on close to every person's desk in every engineering firm. By then, even the most sophisticated general-purpose analysis software was configured for the PC and there was a proliferation of special-use design software for the structural engineer. Throughout the design process, the designer used various software packages in an ad hoc manner, mixing in hand calculations here and there. Perhaps one program was used for frame analysis, another for floor framing, another for checking columns and yet another for the analysis and/or design of foundations.

Development of a third generation of structural engineering software began in earnest in the late 1990s and its adoption has recently become prevalent. This generation integrates analysis and design checks into the same model. In some instances, the designer does not even need to input preliminary sizes since some of the software has design and optimization routines for member selection. Long input and output files are no longer used. The designer builds the model interactively with the software, visualizing the model in three dimensions as it is built. The designer usually has many options for viewing the output, including member-by-member reports, color-coded member stress displays, animated mode shapes and many more. Software is available that allows the designer to model all of the structural systems integrating the analysis and design of floor systems, columns, lateral load systems and foundations. Structural engineering firms see huge productivity gains with such software. It is now practical to provide complete framing plans for the entire building during schematic design and to explore alternate designs without severely impacting the design schedule. Value engineering ideas are tested and implemented late in the design phase or even during construction.

Current Use

The current capability of commercial software to assist the engineer who is analyzing structures is impressive. There are many general-purpose structural analysis programs from which to choose. The most common are comprehensive finite-element programs that analyze two-dimensional and three-dimensional truss, frame and plate structures under static and dynamic loads. The analysis capability goes well beyond that needed for most ordinary building structures. They include advanced non-linear capabilities and many specialty elements that allow the analyst easy creation of any structure imaginable, however complex.

Market pressures on fees for building design are compelling building designers to take full advantage of the productivity gains offered by the automation of today's building design software. The capabilities of this software include automation of framing layout, generation of loads and code-specific load combinations, member selection and optimization, embedded checking for conformance with material design specifications and generation of framing plans.

A Review of Commercial Software for the Design of Building Structures. The January 2005 issue of *Modern Steel Construction* listed 81 software products for the design of structural steel alone.[11] Many are for special purposes such as connection design, design of specialty shapes, vibration serviceability checks, etc. The focus here is on software products intended by their vendors to be comprehensive tools for the design of ordinary buildings (with an emphasis on designer productivity). To provide the reader with a reasonable sense of current capabilities, design products from four vendors whose products are in widespread use throughout the United States are summarized. Capabilities listed for each software package are not exhaustive but rather are selectively listed to convey the complexity and breadth of the current software available to the design profession.

Computers & Structures, Inc. (CSI). Founded in 1975, the development of CSI software spans three decades and began as a result of research at the University of California at Berkeley in the late 1960s. CSI is recognized as a leading producer of software tools for the analysis and design of civil structures. CSI produces multiple software packages tailored

to specific types of structures. SAP2000 — intended for bridges, dams, stadiums, industrial structures and buildings — was introduced over 30 years ago and features a three-dimensional object-based graphical modeling environment. SAP2000 includes a variety of analysis and design features including second order and modal analyses, damper and base isolation, and segmental construction sequencing. ETABS — developed for multistory building structures — provides an integrated building analysis and design environment with a three-dimensional object-based graphical modeling environment. ETABS gives the engineer the ability to model and design moment-resisting frames, braced frames, staggered trusses, rigid and flexible floors, and sloped framing using steel, concrete, composite or steel joist floor systems. SAFE is a special-purpose program for the analysis and design of concrete flat plates and concrete bearing foundation systems, which can also be integrated with ETABS. CSI Detailer can be integrated with the SAFE program to generate comprehensive details of slabs, beams, mats and footings for use in drafting programs.

RAM International (RAM). RAM has been a leader in the development of software solutions that benefit structural engineers, detailers and fabricators since 1988. Their products are designed to aid productivity in the demanding and competitive design market. RAM produces several software packages, each customized for specific applications. RAM Structural System (RAM SS) is a fully integrated suite of building design and analysis software for gravity framing, lateral systems and foundations (including piles). RAM SS is also capable of producing computer-aided design (CAD) drawings for frame elevations, beam and column schedules, floor framing plans and foundation plans. RAM Advanse is a general-purpose, frame-based three-dimensional analysis and design software package capable of designing wood, steel, cold-formed steel and concrete members. RAM Connection is a steel connection design and optimization tool that can run independent connection design checks or run as an integrated component of both RAM SS and RAM Advanse. RAM Concept is a special-purpose finite-element program for reinforced or post-tensioned concrete slab and mat design that is also capable of integrating with RAM SS. RAM CAD Studio works as a component of AutoCAD to help create documents that are linked with models from the RAM SS. RAM Perform is a non-linear analysis package capable of performance-based seismic design and progressive collapse modeling of buildings and other structures.

Research Engineers International (REI). Founded in 1981, REI produces several software packages for specialized applications, but its flagship program is STAAD-Pro 2004 (STAAD). STAAD is an object-based three-dimensional design and analysis tool capable of integrating multiple materials like steel, concrete, timber, aluminum and cold-formed steel into one structural model. STAAD can analyze buildings and bridges using a variety of building codes and it can perform linear or non-linear analyses on the structures. Advanced automatic load generators facilitate easily modeling lateral and gravity loads in complex models. REI integrated the popular steel connection design software package, DESCON, into the 2004 release of STAAD. This component of STAAD produces calculations and design shop drawings for steel connections based on loads generated in the structural model. REI also produces specialized software packages for foundation design, concrete connections and mat foundations.

Rapid Interactive Structural Analysis (RISA) Technologies. RISA Technologies has been an established leader in structural analysis and design software since 1987. RISA produces numerous specialized software packages. Its primary software for the building industry is the RISA Building System, which is composed of RISA-3D, RISA-Floor, RISA-Base and RISA-Foot. RISA-3D is a general analysis package with an object-based three-dimensional graphical interface capable of gravity and lateral load designs for a variety of building materials. RISA-Floor is a design tool that integrates directly with RISA-3D to design floor system gravity framing in buildings. RISABase helps the engineer design steel base plates and anchor rods for steel columns.

RISA-Foot designs and analyzes spread footings for various load conditions. Other RISA products include RISA-Tower for steel truss tower design, RISA-Masonry for masonry walls, columns and beams, and RISA-Section for calculating section properties of custom members.

A Review of Commercial Detailing Software for Buildings. For over 20 years, the steel detailing industry has utilized computers and detailing software programs to increase productivity. During the 1990s, the industry evolved and adopted an approach utilizing three-dimensional models of buildings to manage design information. This three-dimensional information management system helped streamline the detailing process and became indispensable to the construction community. The software programs create a level of efficiency that was not possible before the advent of computers. Project documents that used to take weeks to produce are now being created in a matter of days.

One of the leading steel detailing software packages is SDS/2 by Design Data. Design Data provides multiple products that automate detail creation, control inventory and estimate materials, as well as produce shop fabrication drawings. The connection design software SDS/2 designs connections according to AISC specifications and also produces erection drawings, piece details and complete shop drawings automatically. The software can also track fabrication and erection within the model to better manage resources on large design jobs.

Checking Computer-Aided Designs. Concurrent with software improvements, many experienced engineers who were designing structures in the 1970s and 1980s have developed a concern that the software tools are in the hands of engineers who lack the understanding of the behavior of structures and the methods used by the software. The civil engineering profession has been concerned about the misuse of computer software since the early application of commercial software. Those concerned by this rapidly developing technology have written papers citing case studies of improper use and highlighting the dangers to the profession. The 1991 *ASCE Structures Congress Proceedings*, "Approximate Methods and Verification Procedures of Structural Analysis and Design," and papers by Bell and Liepins, and Emkin, are examples.[12–14] The ASCE Technical Councils on Forensic Engineering, Task Committee on Avoiding Failures Caused by Misuse of Civil Engineering Software prepared a monograph entitled "Guidelines for Avoiding Engineering Failures Due to Computer Misuse."[15]

As cited by Bell and Liepins in 1997, in order to obtain meaningful and accurate results from analysis by computer, the analyst must observe, as a minimum, the following steps:[13]

1. Recognize the important structural actions and include them in the model.
2. Understand the performance of the elements used in the model.
3. Check the input.
4. Check the goodness of the solution obtained.
5. Interpret the computed output.
6. Validate the computed results.

Although the concerns and suggestions cited above are still appropriate today, if not more so, they are focused primarily on the analysis of structures with general-purpose analysis software. The recent, rapid advancements in the automation and integration of analysis, design and drafting in special-purpose building design software has generated a need for engineers to re-examine how to ensure the proper and responsible use of such software.

Guidelines for Quality Assurance & Checking. The six steps put forth by Bell and Liepins are a good place to start. However, these steps can be expanded to address the integrated design capabilities of current building structure design software. The following are suggested guidelines for quality assurance and for the checking of computer-generated designs. The six steps are expanded to include a seventh that relates to quality design. (In addition, commentary on each step highlights the issues pertinent to today's design software.)

Step 1. Recognize the Important Structural Actions & Include Them in the Model. Software

today still requires that the designer idealize the real structure and generate a mathematical model. Designers must understand what structural actions are important to the particular structure given its materials and geometry. Some examples include:

- Many, if not most, software packages incorporate rigid floor diaphragms in their analysis, at least as an option. The designer must decide if this approach is appropriate.
- Are the results sensitive to support flexibility? The source of flexibility might be the soil interface or portions of the foundation outside the model.
- Is the building's lateral load path, from the exterior wall down to the foundation, clearly conceived?
- Are the results sensitive to joint flexibility? In other words, are completely pinned or completely fixed joints appropriate?
- Are important building or element eccentricities appropriately accounted for?
- Is *P*-Delta important and properly accounted for?
- Are thermal, rain or other loads relevant to the design? Recent building codes require explicit design for rain ponding loads. Some jurisdictions require design for volcanic ash fallout.
- Is shrinkage and creep important to the performance of the structure?
- Are there unbalanced earth pressures on the building?
- Do building appurtenances such as stacks, jib cranes, rooftop equipment and flagpoles exert significant loads that should be considered in the model?
- Is the sense of the applied loads correct? (For example, are roof uplift loads acting upward?)
- Does the construction sequence need to be considered in the application of loads in the model?

It is easy for young designers to overlook some of these judgment calls. With the current graphic capabilities incorporating three-dimensional views with extruded, three-dimensional structural shapes, the models can look very "real." However, the computation and design is still based on a mathematical model that approximates the real structure. Fortunately, as designers, it is usually not necessary to model the conditions precisely. Designers can decide to be conservative if the economical impact is minimal. Designers can design for upper and lower bounds for uncertain conditions or for those conditions critical to the structure.

Step 2. Understand the Performance of the Elements Used in the Analytical Model & Know What Design Checks Are Performed & How They Are Executed. Although most software programs have manuals, there are variations in the degree to which they document element performance and the specifics of design checks. Even if the manual states that elements are checked per a particular design specification (such as AISC LRFD, ACI 318), the manuals are silent on a lot of decisions that a designer makes in applying a design specification. Designers should thoroughly understand the design checks performed by the software and the design assumptions coded into the software as well as those that the designer has the latitude to set in the software. Some examples are:

- Understand the software's force sign convention.
- What are the assumed brace point locations?
- How is *P*-Delta accounted for? What loads are used for the *P*?
- How is the shear checked when slabs and drop beams frame into columns?
- How are concrete T-beam properties developed?
- How are short- and long-term concrete deflections calculated? Do the calculations compute cracked section properties and how are they computed?
- Are tension-only braces correctly represented in the model?
- Are deflections local to the member, or do they take into account the deflections of supporting members? (This consideration is particularly important for cantilever members.)
- Are cracked concrete member properties

considered in the characterization of element stiffnesses? (For example, a cracked shear wall will be significantly less stiff that an uncracked one.)

- How are column splices considered in the model?
- Are rigid offsets or other devices utilized to account for frame joint stiffness?
- How does the model address non-prismatic members and members with penetrations or notches?
- Are member forces computed at nodes or at member faces? Does the model allow for setting options in this regard?
- If applicable, how are base plates and anchor rods analyzed and designed?
- Are beam-column joints designed explicitly? There are extensive code provisions that apply to these, especially where seismic design is required.
- Does the model account for induced moments in singly symmetric or non-symmetric sections (for example, single angle or single tee diagonal truss members)?

Step 3. Check the Input & the Input Generated by the Software. Traditionally, input for analysis software could be summarized by these categories: nodal geometry, member incidences and end releases, member properties, material properties, support conditions and loads. It was common practice to print the input file containing these data and check it as part of the quality assurance process. Today's software adds design criteria and design assumptions to the categories of input. Software now has myriad menus the designer needs to open and select options from. Most have defaults that the designer can set and store, and, thus, the designer need not address each and every one of them each time they use the software. However, the designer needs to develop a way to assure that the input selections are appropriate for the particular project. Most software programs today have powerful graphic capabilities that display model information in a three-dimensional view. This capability should be used to review and check the input. Some ideas for checking input are:

- The model's physical properties: general geometry and member organization; member end releases, offsets, rigid end-zones, and member connectivity/connections (i.e., how are crossing members handled?); member sizes, material properties and orientations; and a review of auto-generated meshes of plate elements.
- Boundary conditions: supports, including support conditions — pinned, fixed, springs. Are there any special support conditions warranted by skewed members?
- Loads: Check load input tables, menus, and/or spreadsheets. Understand how the software defines and uses load types (dead loads, live loads, seismic mass, etc.) View load maps on plan views. View concentrated and nodal loads on the model. Check the results of auto-generated loads. Compare total self-weight, total wind shear and total seismic shear with approximate estimates by hand. On a floor-by-floor basis, or other rational categories, compare the total live load and/or total superimposed loads. Check scale factors for seismic design spectra. Check that the software will reverse sense of "plus-minus" loads such as wind or earthquake.
- Check that the software uses appropriate load combinations. Do so by penetrating the software to view the actual combinations and not just checking that the appropriate code or specification is selected. Review if any additional load combinations are required by the applicable material design standards (for example, for reinforced concrete, the Massachusetts building code requires the worst combination from its Chapter 16 combinations and those of the ACI 318 combinations). Review if custom load combinations are generated for unique conditions (like Massachusetts' exceptions for dead load factors, etc.).
- Global settings: units. This selection may be a global setting or it may vary from one dialogue box to another. Code checking criteria: LRFD versus ASD, U.S. versus British standards, etc. Live load reduction code and related parameters.

- Design constraints and design criteria settings. These settings are often hidden from the "screen side" of the software. Depending on the software, there can be many different settings and defaults spread over many different dialogue boxes. An incorrect or inconsistent setting can have a significant impact on the software's output. A few illustrative examples include: composite or non-composite construction; shored or unshored construction; deflection limitations, camber limitations; shear stud lengths and diameters; concrete material strengths; maximum and minimum reinforced concrete reinforcement ratios for different elements; the extent to which slab systems brace tops of beams against lateral torsional buckling; and design-check settings (such as whether gravity loads should be considered or not in the calculation of the shear strength of a reinforced concrete shear wall).
- Check for coordination with the architect and other disciplines for: openings, stair loads, exterior wall loads, and suspended and floor-supported equipment loads.

Step 4. Check the Goodness of the Solution. Whether using the software for complex analysis or for the analysis and design of less complicated structures, the results and output should be inspected for software error messages and warnings. Take the time to understand any messages. It may turn out that certain warnings do not impact results, but this must not be presumed. Chase down each and every one of them and eliminate them unless there is great certainty that they do not impact the results.

Step 5. Interpret the Computed Output & the Design Check Reports. The design output may require interpretation and/or further calculations to arrive at meaningful results. Many times, with the new generation of building design tools, this step is simply understanding and reviewing software messages for members that do not meet design checks, or viewing color coding of members that represent demand-to-capacity ratios. However, stress output for plate elements may need more work and interpretation to understand which stresses are being reported and which ones require a meaningful comparison to code-allowable stresses. Some considerations would be:

- Are extreme fiber stresses being reported or average stresses?
- Do orthogonal stresses need to be combined to get principal stresses?
- How do plate elements compute and report out-of plane shears?
- Review deflection reports: Are corner or center of mass drifts reported?

Step 6. Validate the Computed Results. The results must be assessed for validity and for any signs of unexpected response of the structure and unexpected design results. Unusual or unexpected results should be thoroughly explored until the issue is understood or the error causing the behavior is found. Some suggestions and examples for validating results are:

- Review reactions: Review locations with zero reactions and review if these represent an error or not. Review reactions for individual load cases. Do the relative magnitudes of reactions for the individual load cases make sense for the various support locations? Review individual support locations. Do the magnitudes for various load cases make sense for a given support location? Are the reactions for a given load case in equilibrium with the applied loads? Are there high shears where they are expected? Are there high overturning forces where they are expected?
- Review the deflected shapes: Do the deflected shapes make sense for the individual load cases and the intended model? Do the mode shapes make sense for the intended model? Do the periods make sense? How does the fundamental period for a building structure match code approximations? Do the deformations compare well with estimates by hand or with experience with similar structures? Look at the deflected shapes in both plan and elevation views as well as

in three-dimensional views. Review the center of mass and center of rigidity. If the software reports the center of mass and the center of rigidity, do they make sense? Do they seem rational given the deflected shapes?

- Review story shears and individual frame shears: Do the story shears relative to one another make sense? Do the story shears sum to expected base shear estimates? Review the impact of diagonal members that span more than one story. Review the frame shear (or wall, or brace) distribution at each floor.
- Spot check member forces: Look for members with zero forces and review if these are the result of an error or not. Review flexural members to verify that shears and moments act in the axes expected. For representative members, do the member forces make sense for the various load cases?
- Review member design reports: Review critical and representative member detailed design reports. Review if the design accounts for special seismic requirements such as the out-of-plane bracing of brace joints, amplified loads for member connections or for member design, etc.

Step 7. Review the Quality of the Design. Review the design for general quality. The current generation of software does not guide users with respect to the following considerations. Assess the quality of the design with respect to these considerations:

- Is the design rational? Are the member sizes reasonable and do they fit into norms for similar buildings? If not, understand what is special about this project.
- Is the design appropriately redundant and robust?
- Did the design appropriately address serviceability (for example, ponding, vibration limits)?
- Do the deflection limits, including short- and long-term dead loads and live loads, reflect the architectural and mechanical details?
- Does the design address durability appropriately?
- Does the design address penetrations of walls, beam, etc., for openings for other trades?
- Does the design maximize regularity, repetition and simplicity?
- Are the structural costs within budget and appropriate for the building type and project requirements?
- Does the design of the foundations size the foundations using service loads and allowable bearing pressures? Does the design of concrete foundation members factor the loads for LRFD design?
- Does the design address structural checks not addressed by the software? Examples often include: diaphragm strengths and stiffness and the effects of large openings on same; local wind loads on columns; torsional effect from eccentrically placed exterior loads; load path from the diaphragm to the lateral-load-resisting members (for example, is the load path viable and are the in-plane design forces accounted for?); and combinations of wind uplift and shears on deck diaphragms.

Modeling & Design Pitfalls to Look Out For. Despite recent advances in the capabilities of software packages, there remains the need for care to keep modeling, analysis and design from deviating from the reality of the built structure. Potentially serious problems can ensue from not properly conceiving or interpreting design models. The following are but a few examples of pitfalls that, for now, can only be avoided by the exercise of judgment by an experienced designer:

- Rigid elements introduced to model zones that are a lot stiffer than the bulk of the model can introduce numerical instabilities. For example, a designer often introduces a rigid member to model the connection between a beam element and a line element for, say, a concrete wall. The assignment of the stiffness must take into consideration the potential for numerical instability arising from some entries in a stiffness matrix being much larger than other entries. These large differences in

magnitude will introduce numerical errors in the decomposition of the stiffness matrix. The potential for this problem will vary with the complexity of the structure, the software code and the precision of the computer hardware. Sometimes, analysis results that are orders of magnitude beyond the expected can be a sign of a numerical instability. Other times, numerical instabilities are subtly manifested and might go undetected.

- Assignment of material properties must take into account construction practices. A concrete frame analysis that assigns high-strength concrete to the column and beam elements might miss the fact that, unless otherwise specified, the concrete strengths of the beams will be the same as that of the floor system.
- Current floor system design software does not handle slab depressions well. Large slab depressions can have the effect of invalidating otherwise valid rigid diaphragm modeling assumptions and effective width assumptions for T-beams and composite beams.
- Software packages that model rigid diaphragms, unless otherwise directed, will usually "lock" the deformations of the members within the diaphragm. Members within the diaphragm are thus modeled as having no axial deformation or axial forces. There are certain members (such as collector members) that deliver axial forces to a braced frame and beam elements in a chevron brace where this modeling assumption is unconservative.
- Software packages that model continuous floor systems (such as reinforced concrete slabs and beams, like ETABS), with shell or plate finite elements for the slabs and line elements for the beams require a thorough understanding of how the finite-element mesh interacts with the beam elements for the transmission of out-of-plane and in-plane forces.
- Design of composite systems with hanging loads should account for the construction sequence: Do the hanging loads occur before or after steel-concrete composite action?
- Uplift in beams is often overlooked in programs that have settings that continuously brace the top flange of a beam. Net uplift forces will produce compression in bottom flanges that require attention by the designer perhaps with a separate "fake" loading case with the net uplift load and the deck-braces-flange setting turned off.
- Seemingly small design changes can have a dramatic impact on the behavior of a building. A rectangular building with two short sides that counts on a below-grade foundation wall to transfer lateral loads to the ground could see its behavior dramatically altered if a significant portion of one or both of the short foundation walls becomes an areaway.
- P-Δ calculations should use loads that are consistent with the material code second order moment amplification provisions.
- Can load combinations be customized? Certain software packages allow customization of load combinations that allow the designer to use the 1.3 dead load factor in the current edition of the Massachusetts building code. When customized load combinations are not possible, factoring up the dead loads might be necessary. The designer must keep track of these increased dead loads for their impact on foundations, deflection checks, etc.

Verification of Commercial Software. An issue facing the structural design profession is the verification of commercial design software. The steps outlined above for quality assurance and checking help to verify the results for a particular design. For most common building design projects, these steps, combined with sound engineering judgment and experience, are adequate to uncover deficiencies or errors in the software that may surface due to that project's specific parameters. However, designers must still place considerable reliance on the software doing what it claims to do without error. Designers are operating under the assumption that any errors of significance to the design will be detected during their review and validation of the results.

Thus, designers become comfortable with particular software over time as it is used, and the verification of software is limited to the validation of results on a project-by-project basis.

Transparency Issues. Generally speaking, unless the software is open source, its code is not available to the public. Even if it were, most practitioners do not have the programming experience and skill to interpret the code in order to verify its operation and application. What designers know about the operations and function of software is limited to what is published with the software and what they can infer from the available output reports. Designers should look for software that provides detailed explanations of operations and functions as well as output reports that provide step-by-step member capacity development and code check results and not just a final stress or demand/capacity ratio.

Responsibilities of Vendors & Users. There are no current standards or codified requirements for structural design software. Organizations such as the Software Engineering Institute (SEI), the Centre for Advanced Software Engineering (CASE) and the National Council of Structural Engineers Association (NCSEA) should partner with leading structural design software companies to explore ideas about guidelines and/or standards for qualifying and verifying software claims on operation and function. Perhaps the engineering industry can develop a voluntary certification program where software can receive certification after thorough review by both design experts and computer programmers. In addition, there should be a program, analogous to the American Institute of Steel Construction (AISC) certification for fabricators, where software companies are certified as maintaining good practice with respect to quality assurance and documentation.

Documentation of Computer-Aided Design. With increasing reliance on design software, design documentation is another issue for designers. Construction drawings and specifications document the final design. However, the calculations document the design adequacy. Policies about design documentation and the preservation of the design calculations vary among structural design firms. Some archive the calculations with the drawings and specifications; others destroy the calculations after a specified period of time; and some destroy the calculations as soon as the building receives occupancy. Each of the various policies has its pros and cons, but many firms would agree that the design calculations, or other design adequacy documentation, must be kept at least through the firm's checking process and quality assurance program.

As engineers rely more and more on integrated analysis and design (and produce fewer and fewer hand calculations), engineers must decide how to document the design checks using input reports, graphic output and design check reports from the software. Therefore, the design documentation becomes tailored, in part, by the specific piece of software. Each firm should have a quality management program that includes guidelines on how to document the design adequacy when using design software. If a firm believes it is important to archive calculations, then decisions must be made about storing them in paper or electronic form. If storing in electronic form, the fact that software goes through version evolution and new versions are not always backward compatible must be taken into account. Perhaps it is best to keep text files documenting the input, results and design adequacy.

The following are general suggestions for computer design documentation:

- Note the name and version number of the software.
- List the software default settings for the project (including the design defaults as well as model and analysis defaults).
- Document the model (including geometry, member and material properties, and support conditions).
- Note mass, loads and load combinations.
- List the design codes and software design settings used.
- Record the analytical results (such as reactions, story shears, story deformations, etc.).
- Record member design checks, at least for the controlling load cases.
- Keep records of the result verification process.

Keeping a log of the evolution of the model used is another good idea for documentation. The model log can record the changes, and the reasons for the changes, to the model throughout the design process.

Impact on Staff & Staff Training. Management of today's design firms must address training issues in light of the automated nature of the software. For example, young designers do not become familiar with member design specifications by frequent use as was the case just a decade ago. The skill sets needed to design with current software are much different than the skill sets needed when design was done by hand calculations. Designs are now accomplished using automated software without the user necessarily being intimately familiar with the design standards and, perhaps more importantly, without the user being familiar with the fundamental philosophies embedded in the design standards. When the design checks were performed by hand, practitioners read the standards, codes and user commentaries before performing the calculations. In addition, industry material organizations (such as ACI, PCI and AISC) provided design guidelines and sample solutions to aid and teach the designer. With today's automated design checks, young engineers need explicit training on the design codes and standards since they may not become familiar with them through routinely performed hand calculations.

Future Trends

The leading software vendors will continue to advance their structural design software. Most have, or are developing, a system or suite of software so that the designer can become familiar with one vendor's tools and use them for various structural materials and structural systems. The vision of the vendors is to have interoperability within their suite of software. For example, the designer will model the entire building in a modeling module, and use other software modules from the vendor to design foundations, frames, shear walls, flat plates, connections, base plates, etc., without further input of loads or structural geometry. In addition, the vendors will move to integrate the drawing production process with their suite of software. The leading vendors already have tools, with varying degrees of sophistication, for exporting data from their software to CAD systems.

Parallel with the development of integrated analysis and design software was the development of detailing and fabrication software for the structural steel industry. Detailers can now build their own three-dimensional models from the design drawings and use specialty software to prepare detailed shop drawings and, in some instances, convey computer files directly to the fabrication equipment. AISC is promoting electronic data interchange (EDI) technology to allow the exchange of data from the design software to the fabrication software. This interchange is only one piece of a more comprehensive concept of interoperability in the construction process.

According to the National Institute of Standards and Technology (NIST), interoperability relates to both the exchange and management of electronic information.[16] Individuals and systems are able to identify and access information seamlessly, as well as comprehend and integrate information across multiple software systems.[16] The manufacturing sector, as well as financial centers and medical centers, have created interoperability standards and realized efficiencies in operations and service. The construction industry has not yet developed similar standards. Reasons for the construction industry lagging behind may include:

- relative to the manufacturing and financial sectors, the construction industry is made up of many more and smaller entities without resources to affect the industry; and,
- relationships between owners, designers and contractors inhibit communication and teamwork, resulting in fragmentation of the process.

NIST funded a study entitled "Cost Analysis of Inadequate Interoperability in the U.S. Capital Facilities Industry."[17] The study quantified the cost of inefficient interoperability in commercial, institutional and industrial facilities in the year 2002. Results showed that inef-

ficient interoperability cost the industry $15.8 billion in 2002 with increased new construction costs of about $6 per square foot.[17]

The use of computers for structural engineering will rapidly go beyond integrating analysis and design. Because of the potential productivity gains implied by the NIST estimate of $15.8 billion of waste, the industry will move towards interoperability, if not by national standards, then by the dominance of commercial leaders. De facto standards have been set by the dominance of commercial leaders (such as those set by Autodesk with AutoCAD and the .dwg CAD file standard). Interoperability will come to the construction industry through highly detailed information databases for entire buildings that contain the information for all the materials and systems that comprise the building. These relational databases are called building information modeling (BIM). Autodesk and Bentley, as well as other companies, have already released such products incorporating BIM.

BIM uses the concept of object-oriented three-dimensional modeling that is already being employed by the aerospace and automobile industries. In other words, the user works with intelligent components (such as walls, doors and windows, columns, footings, ducts, etc.) to create a virtual building from which plans, elevations, sections and details are extracted in the form of drawings. The drawings look like those developed from traditional two-dimensional CAD, but they are in fact just reports from the database. Building data such as material quantities can also be extracted in the form of schedules. The object-oriented modeling allows for the rapid construction of the model.

The leading BIM software solutions also allow bidirectional editing. Changes to the extracted views or schedules affect the model as well as all other views, schedules and information throughout the model. All information is interrelated at all times in one database system. The fact that the construction documents are reports from the database ensures that they are well coordinated at least with respect to the data in the model.

All disciplines of the design team, as well as the contractor, can collaborate on the development of the database and the model. With all the data from all the parties entered into one database, the information on the project is well coordinated. Programming embedded in the software identifies interferences or conflicts between the trades and disciplines. But BIM moves beyond just the physical representation of the building and the transformation of paper documents into electronic form. The database contains information that can assist cost estimates, scheduling, energy analysis, fabrication and maintenance.

BIM database solutions, such as Bentley Architecture and Revit (from Autodesk), are already developing relationships with structural analysis and design software to achieve interoperability with them. Similar developments are underway for mechanical engineers as well. The vision is that the model developed in BIM will include all pertinent engineering data. Data then flows from BIM to the analysis and design software with results coming back to update BIM. Proponents of the BIM concept see this new process as a means to reduce risk from the now-fragmented process that relies on the architect (or design professional leading the project) to coordinate and integrate work from several separate entities.

The value of these first-generation BIM solutions will be the coordination of data contributed from fragmented sources (architects and consultants) by providing interference checks, allowing clearer communication between participating parties and producing drawings and documents from the same database or interrelated databases. However, just as structural design software has become more intelligent and comprehensive, so too will BIM become more capable — containing or associating with analytical tools and other software, and, if not automating the design process, certainly automating the design documentation process.

No one knows the ultimate impact of BIM software on the professional services market. Outsourcing of engineering man-hours has been in the press lately. The premise for outsourcing is that overseas labor is becoming well trained and is in a position to provide engineering hours for considerably less

money. On the one hand, as design software reduces man-hours, the financial gains from outsourcing diminish. On the other hand, automated design software homogenizes the product, enabling overseas manpower to produce exactly what is done here. The ability to meet and interface with the rest of the design team to exchange and integrate design ideas becomes the major value of using a local team. What will BIM do to this equation? Will this technology make it easier for overseas engineers to contribute locally?

Conclusion

The first and second generations of structural software led to modest productivity gains as these analysis tools had little impact on the overall design process, especially with respect to how the structural engineer interacted with the rest of the design team. The third generation of structural software in use now, by integrating the analysis and design of building-whole systems, has had a major impact on the design process. It is debatable whether structural engineers have yet fully adapted best-practice procedures for software that can almost fully automate the design and analysis of a building. And as structrual software moves towards its fourth generation and is tied to BIM software that allows the automatic generation of drawings and reports, engineers will have to redefine their roles and responsibilities for what well may be a paradigm shift in the industry.

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Tunneling Through Soft Ground Using Ground Freezing

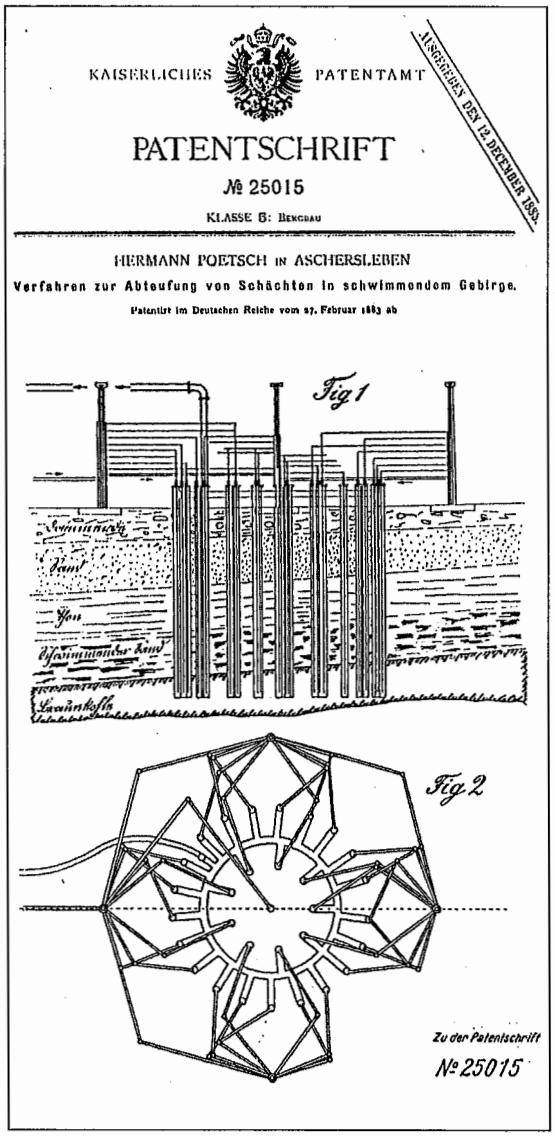
Ground freezing represents a viable solution to create a watertight and load-carrying soil body in water-bearing soil.

HELMUT HASS

The ground freezing method was developed in the nineteenth century by the German engineer Friedrich Poetsch. The method was developed to sink shafts through water-bearing soils down to hard rock and coal seams. It was the only safe method to construct shafts with depths of more than 50 meters (160 feet) in water-saturated soil. The deepest freezing shaft completed so far in Germany was in Rheinberg with a depth of more than 600 meters (1,925 feet). Figure 1 shows the original patent document for this method, which was dated 1883.

Ground freezing is a process where in-situ pore water is converted to ice. Like the cement in concrete, the ice bonds the soil particles together, imparting strength and impermeability to the frozen soil mass. Ground freezing is based on withdrawing heat from the soil. A continuous supply of energy is usually required to establish and maintain a frozen soil body.

In order to freeze a body of soil, a row of vertical, horizontal or inclined freeze pipes have to be drilled into place. An open-ended inner pipe (sometimes referred to as the down-pipe) is inserted into the center of the closed-end freeze pipe (see Figure 2). The down pipe is used to supply the freeze pipe with a cooling medium, usually brine or liquid nitrogen. The inner pipe is connected to the supply line and the outer pipe to a return line (when brine is used) or an exhaust line (when liquid nitrogen is used). The coolant flows through the inner pipe to its deepest point. On its way back through the annulus between the inner (down) pipe and freeze pipe, the coolant picks up heat from the soil and warms. Due to the flow of the coolant, the frost penetrates outward from the pipe into the soil and a ring of frozen soil occurs around the freeze pipes as heat is withdrawn. Freezing generally occurs initially at the bottom depths and progresses upward. Depending on the arrangement of the freeze pipes, all shapes of frozen soil walls (bodies) can be achieved as required for an individual task.


KAISERLICHES PATENTAMT

AUSGEGEBEN DEN 12 DECEMBER 1883.

PATENTSCHRIFT

№ 25015

KLASSE 5: BERGBAU

HERMANN POETSCH IN ASCHERSLEBEN

Verfahren zur Abteufung von Schächten in schwimmendem Gebirge.

Patentirt im Deutschen Reiche vom 27. Februar 1883 ab

Fig 1

Fig 2

Zu der Patentschrift

№ 25015

FIGURE 1. A ground freezing patent document from 1883.

Brine freezing requires a closed circulation system and the use of refrigeration plants. The brine (usually water mixed with calcium chloride, $CaCl_2$) picks up heat at the bottom of the down pipe, then it flows back through the insulated surface manifold system before returning to the freeze plant for recooling in the vaporizer. The principle of brine freezing circulation is presented in Figure 3. The brine supply temperature, *T*, generally ranges from –20 to –37 degrees Centigrade (–4 to –35 degrees Fahrenheit).

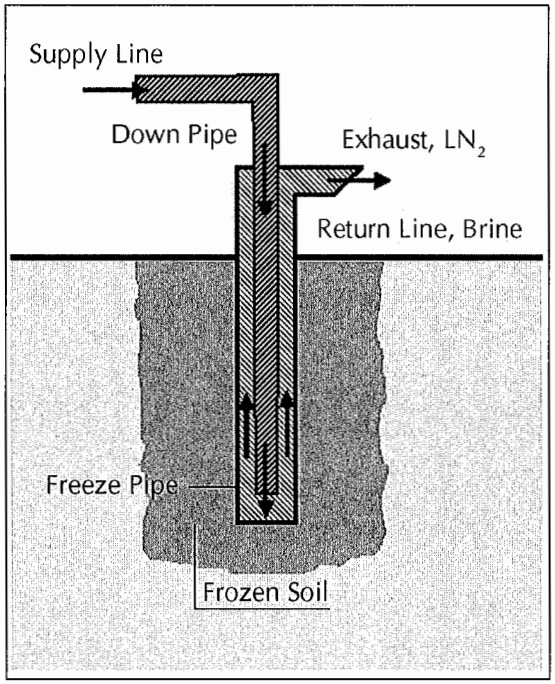


FIGURE 2. A diagram showing the ground freezing principle.

The entire freezing plant consists of the required number of freeze units, several additional components (such as low voltage switch-gears), tanks for the brine backflow and the back cooling units (cooler) that remove heat from the warmed cooling agent. Several freeze units can be combined in a more powerful freeze plant. To minimize fresh water consumption, special recooling systems should be connected for heat exchange with the air. Currently, it is state of the art to use ammonia as the cooling agent to remove the heat. Ammonia is much more environmentally friendly than hydrocarbon fluoride (Freon), which was used until 20 years ago.

With liquid nitrogen freezing, heat is extracted from the soil through direct vaporization of a cryogenic fluid (LN_2) in the freeze pipes. From an on-site storage tank or directly from a tank truck, the LN_2 is fed through an insulated surface manifold system, usually consisting of copper pipes and quick-connect cryogenic hoses, into the inner pipes as shown in Figure 4.

The LN_2 starts to vaporize at a temperature of –196 degrees Centigrade (–321 degrees

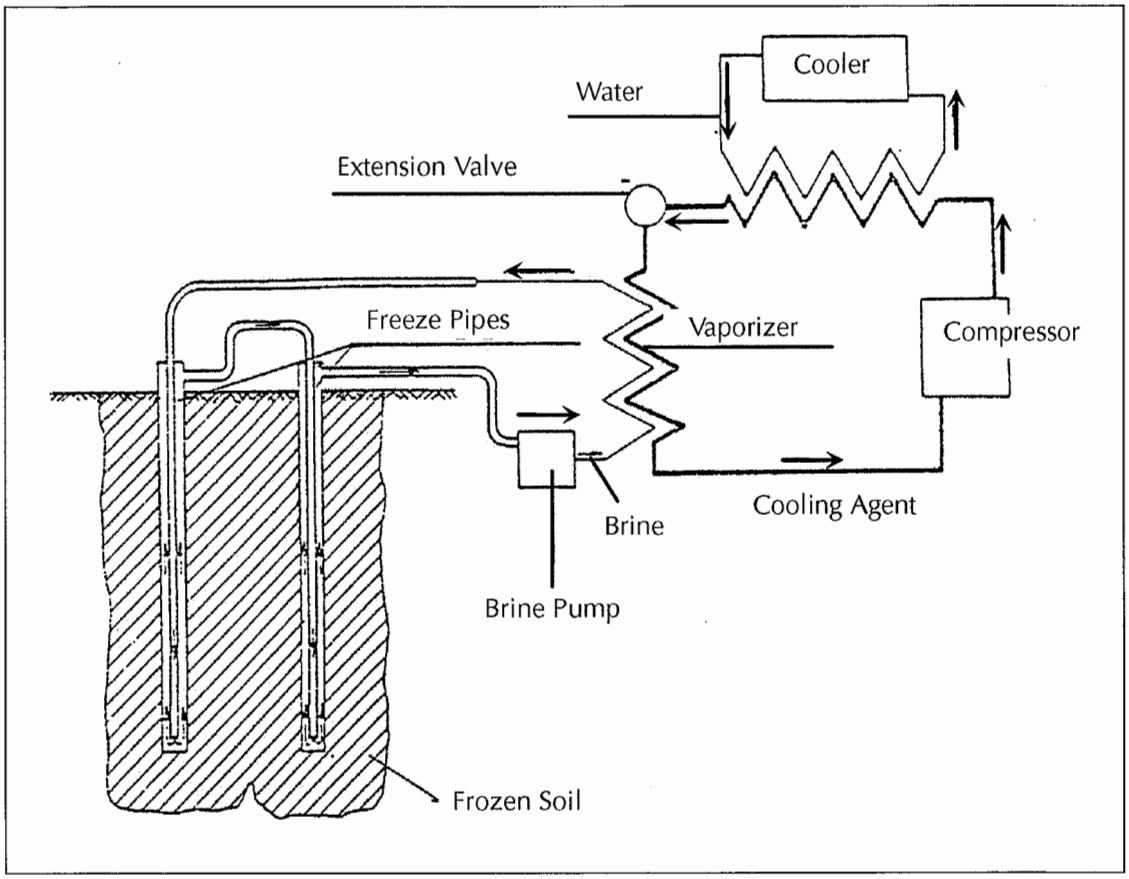


FIGURE 3. Typical set-up for brine freezing.

Fahrenheit) in the annulus between the freeze and inner pipes, picking up heat on its way up. The cold nitrogen gas is directly vented into the atmosphere; the gas exhaust temperature is measured with temperature sensors. The amount of LN_2 that is fed into the inner pipe is controlled by a cryogenic two-way solenoid valve. The solenoid valve is controlled by the nitrogen gas exhaust temperature, and either opens or closes based on preset temperature limits.

Freezing with LN_2 is fast. A frozen soil body can be formed within a matter of a few days with LN_2; whereas it takes weeks for the brine freezing system. However, due to its high cost, the use of LN_2 for ground freezing is usually limited to short-term applications or limited volumes of frozen soil.

The Behavior of Frozen Soil

The behavior of frozen soil under quasi-static loading usually differs significantly from that of unfrozen soil due to the presence of ice and unfrozen water films. Freezing will increase the strength and stiffness of the soil. However, frozen soils are much more subject to creep and relaxation effects and their behavior is strongly affected by temperature changes. The viscoelastic behavior of ice is dependent on many additional factors, such as salinity, pressure, strain rate, crystal orientation and density.

Time-Dependent Creep Behavior. Figure 5 shows typical idealized curves of creep and corresponding strain rate versus time, assuming constant stress and isothermal conditions. Three distinct phases, or stages, of creep are usually evident. After an instantaneous strain (elastic deformation), the primary phase, or Stage 1, is characterized by strengthening with a continuously decreasing strain rate. The secondary or steady-state creep phase, or Stage 2,

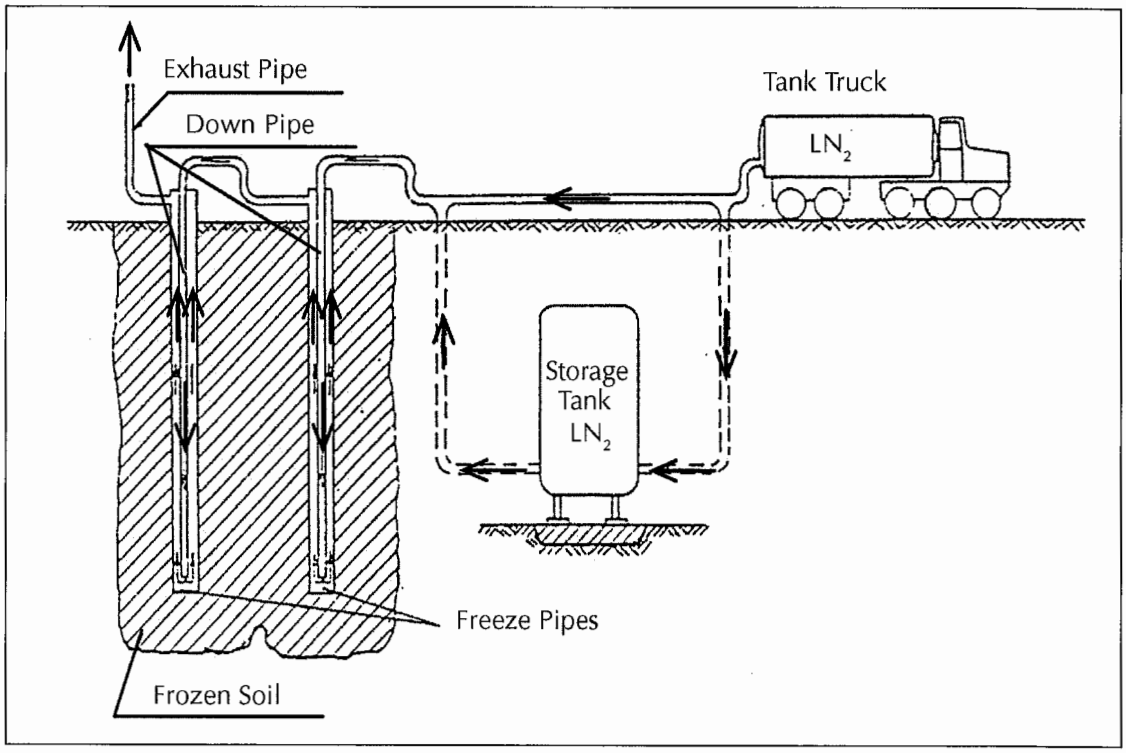


FIGURE 4. Typical set-up for liquid nitrogen freezing.

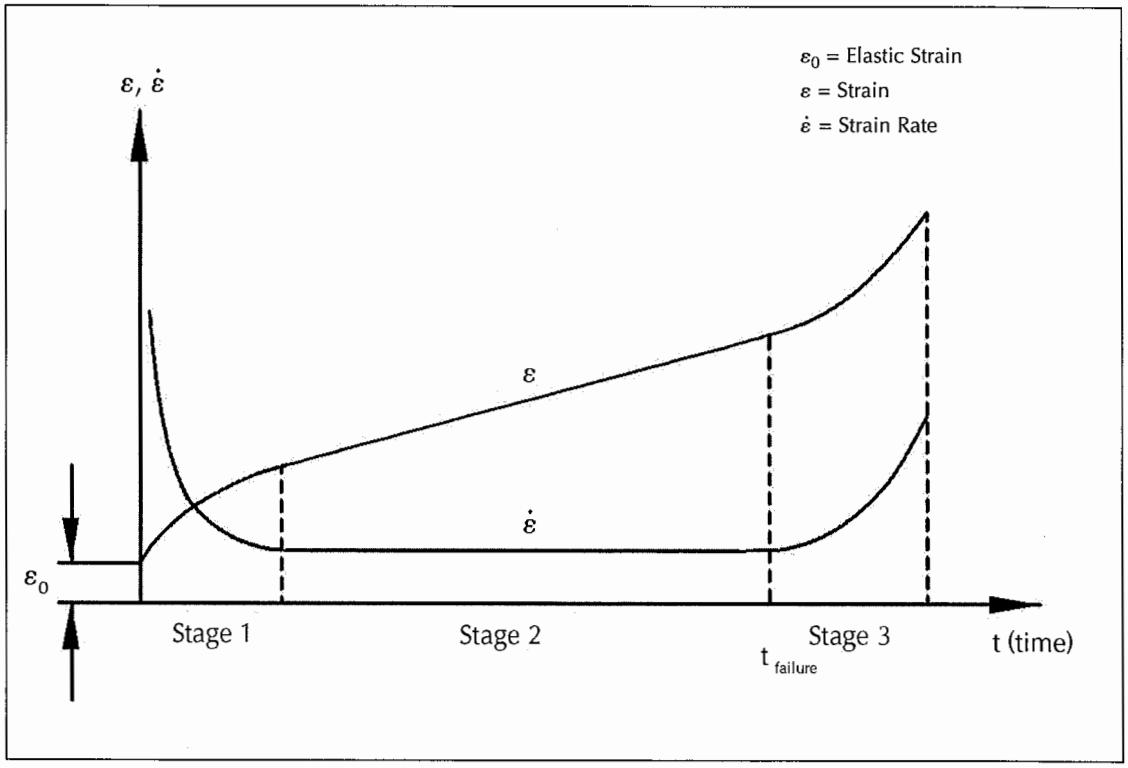


FIGURE 5. A typical creep curve for frozen soil.

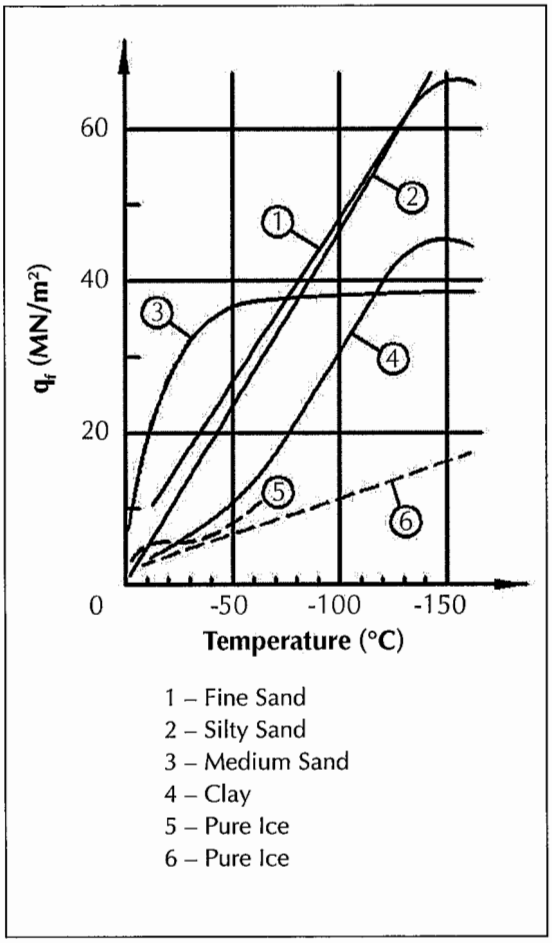


FIGURE 6. Frozen soil strength versus temperature.

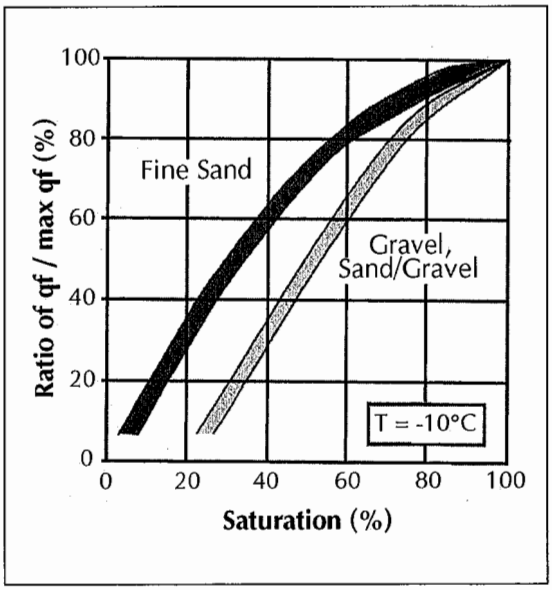


FIGURE 7. Frozen soil strength versus saturation.

is characterized by a constant creep rate, which is the minimum rate reached during the test. Finally, the tertiary phase, or Stage 3, is characterized by an accelerated creep rate, which leads to the ultimate failure of the specimen. Because of the creep behavior of frozen soil, a decrease of strength and stiffness from 40 to 60 percent of the initial values has to be considered, depending on the time period of ice service on a project.

Temperature-Related Behavior. The uniaxial unconfined compressive strength, q_f, of the frozen soil is an important value for the structural design, as is the Young's modulus of elasticity, E. These parameters are not only time dependent, they are also strongly dependent on the temperature. In Figure 6, the uniaxial strength versus temperature is shown for specific soil layer tests conducted in the lab on a range of typical soil types and for pure ice. (The curves shown in Figure 6 are based on lab test results from a project in Düsseldorf, Germany.)

The average temperature of frozen soil usually ranges from –5 to –25 degrees Centigrade (23 to –13 degrees Fahrenheit) and the lowest temperature is usually not colder than –50 degrees Centigrade (–58 degrees Fahrenheit). In this range, the strength behavior of frozen soil is almost linear.

Influence of Water Saturation. The strength and the stiffness of the frozen soil also depend on the degree of water saturation as the frozen water bonds the soil particles. Figure 7 shows the compressive strength of frozen soil versus the degree of water saturation. With a saturation of 40 percent, fine sand has a compressive strength of approximately 60 percent of its maximum strength (a saturation of 100 percent). Given the same saturation, gravel achieves approximately 35 percent of its maximum value.

Frost Heave. Another property that has to be considered is the frost heaving of frost-susceptible soil. Figure 8 shows typical curves of frost heave rate versus permeability. Frost heave is reduced as overburden pressures

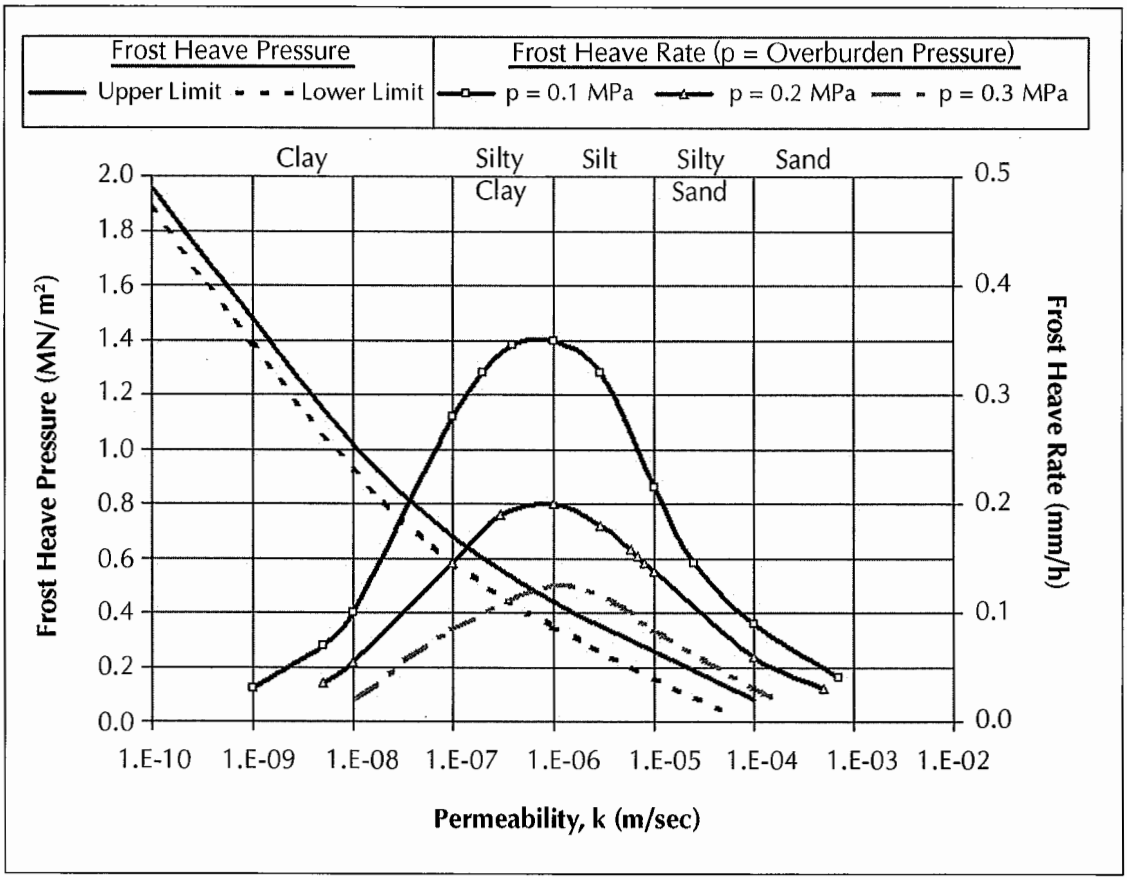


FIGURE 8. Frozen soil frost heave behavior.

increase. Therefore, three curves for different pressures are shown. The diagram also shows curves of possible frost pressure that may occur if the soil is confined due to local conditions. It is notable that the highest frost heave occurs in soil with a permeability of 1×10^{-6} to 1×10^{-7} meters (0.00004 to 0.000004 inches) per second. This permeability is typical for silt or silty clay. The highest frost pressure can occur in clayey soils.

Groundwater Velocity. Groundwater flow can have a major impact on the freezing operation. Flowing water provides a continuous source of heat, and can significantly delay the freezing time. In a worst-case scenario, a state of thermal equilibrium can be reached in which the soils stop freezing and closure (merging of the frozen soil cylinders of adjacent freeze pipes) of the freeze wall cannot be achieved.

As a rule of thumb, effective flow velocities (seepage velocity) of less than 2 meters (6.5 feet) per day for brine freezing and 4 to 6 meters (13 to 19 feet) per day for LN_2 freezing seem to have little or no effect on the freeze wall development. For higher groundwater flow rates, the following measures can help ensure timely formation of the freeze wall:

- reduced freeze pipe spacing;
- installation of additional freeze pipes (second or third row) preferably on the upstream side;
- lowering of the brine temperature through increased refrigeration capacity;
- use of liquid nitrogen in critical areas;
- grouting to reduce permeability and groundwater flows to acceptable levels; and,
- installation of intercepting wells to reduce the groundwater gradient.

Application of Ground Freezing

Ground freezing is mostly used for temporary

ground support or as temporary structural elements or as a groundwater control system. The advantage of frozen ground is that frozen water is 100 percent impermeable soil. Obstacles like stones, concrete remnants or similar materials, which cause problems when grouting techniques are used for sealing tasks, will just be embedded in the frozen soil volume as the frost grows through and around the obstacles.

Ground freezing is being used in underground construction projects for:

- sinking and lining of deep mineshafts to depths of more than 600 meters (1,925 feet);
- deep excavations (shafts);
- tunneling using the sequential excavation method (SEM) under the protection of a structural and watertight frozen soil body;
- cross-passages between shafts and tunnel tubes, or between tunnel tubes;
- large open excavations, retaining walls;
- temporary soil improvement under foundations;
- temporary sealing of leaks; and,
- temporary water cut-off for connections at the interface between existing and new underground structures.

Ground freezing is environmentally friendly since no barriers (such as diaphragm walls, chemical grouts or other grouting materials) remain in the ground after the application. Ground freezing can also be used for environmental applications such as freezing the soil of a landfill to keep the contaminated material in the soil from spreading during excavation.

Investigation & Lab Testing

Groundwater Conditions. Groundwater flow can have a major impact on the freezing operation. The hydraulic conductivity, *k*, and the groundwater gradient on the site must be ascertained to evaluate any existing groundwater flow conditions. Attention must be paid to possibly increased flow velocities in localized zones of coarse-grained layers. Existing or planned nearby dewatering measures may also influence the groundwater flow and may cause high groundwater velocities.

Observation wells upstream and downstream of the frozen soil body have to be installed. Investigation of the existing groundwater flow conditions can be conducted by using single borehole testing in observation wells. These tests should encompass:

- Dilution tests using dyes and other tracers. With this test, the groundwater flow direction and velocity can be determined for each layer versus depth.
- Determination of the permeability over the whole depth with flowmeter tests.

In case tracer tests cannot be performed, the permeability of each layer should be determined by using flowmeter tests and should be based on grain size distribution. Based on the groundwater level gradient, the flow direction and velocity can be estimated.

Subsurface Conditions. It is strongly recommended to investigate the existing soil conditions by using drilling methods with continuous undisturbed sampling to get a continued soil profile over depth that reveals all existing layers. The core diameter should be at least 50 millimeters (2 inches), but sizes of 75 to 100 millimeters (3 to 4 inches) are preferred.

Laboratory Testing Program. A test program for ground freezing purposes is required to verify the estimated soil parameter values or to determine the actual range of soil parameters in frozen and thawed conditions. The test program has to be conducted by experienced personnel in a laboratory specially equipped to evaluate frozen samples.

The test program should provide at least the following information about the soil layers at the construction site:

- Index properties: grain size distribution, specific gravity dry unit weight, Atterberg limits, degree of saturation, salinity;
- Frost deformation behavior in case the soil is frost-susceptible (frost heave tests); and,
- Geotechnical soil properties on unfrozen, frozen and thawed samples. These tests should be performed on soils taken from the boreholes at the site to determine: temperature-dependent shear strength,

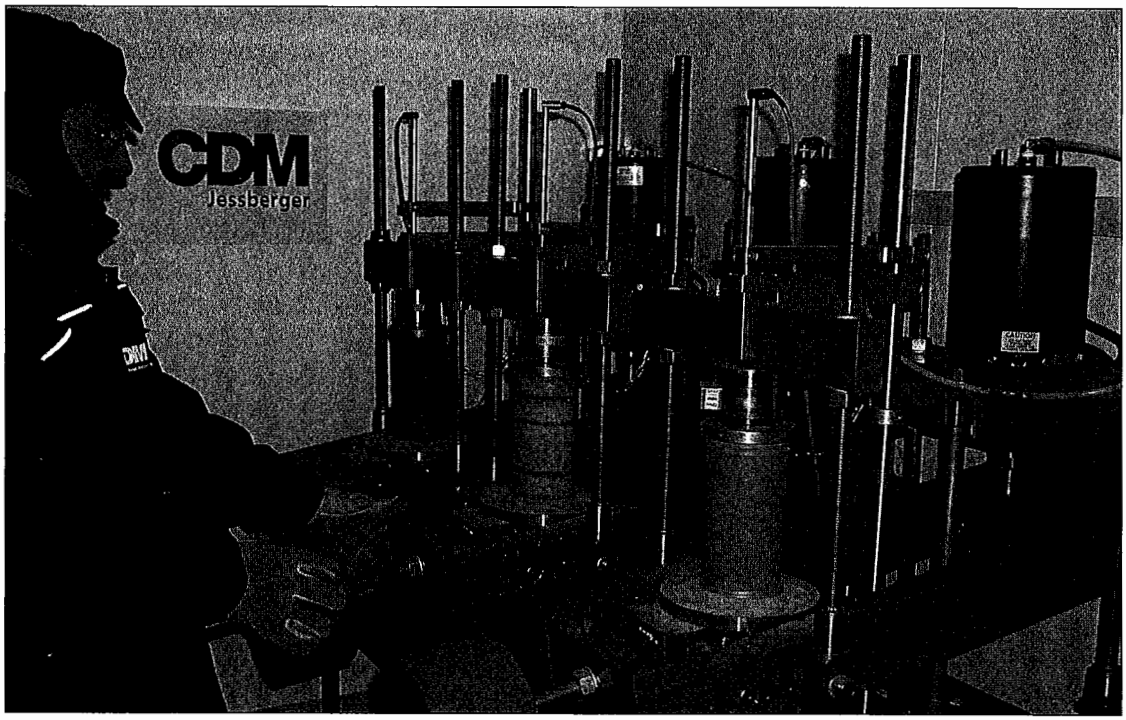


FIGURE 9. Frozen creep tests in a special laboratory.

temperature-dependent unconfined compression strength, and time- and temperature-dependent creep behavior (see Figure 9).

Based on the results of the lab test program, the temperature- and time-dependent frozen soil design parameters can be specified as input data for the final design.

Design of Ground Freezing Applications

Thermal and structural calculations are required for the design of a ground freezing project. The thermal design determines the freezing time to form the freeze wall, freeze plant capacity, freeze plant operation during maintenance freezing and temperature development, as well as temperature distribution in the soils. The structural design provides the dimensions of the freeze wall and the required average freeze wall temperature.

Thermal Design. The thermal design can be based on rough calculations for the freeze wall growth and the heat flux using analytical methods during a pre-design phase. In most final designs, thermal calculations using the finite-element method (FEM) are required to verify results of the pre-design and to optimize the freeze pipe spacing and arrangement as well as freeze plant capacity. Using this method, the actual conditions (freeze pipe spacing and location, interrelation of adjacent freeze pipes, dependencies of freeze wall growth between different layers, etc.) can be effectively varied and optimized. For the actual thermocouple locations, these calculations can predict time-dependent temperatures that can then be used for direct comparison with the actual thermocouple readings during the freezing operation.

Using the numerical finite-element method, the following design conditions can be considered:

- different soil layers;
- different initial temperatures in the soil layers;
- varying freeze pipe spacing and varying freeze pipe temperatures;
- different and temporarily changing freeze plant capacities for brine freezing;

- intermittent freezing during maintenance; and,
- additional heat sources or thermal boundary conditions.

The results of the analyses should include:

- the time-dependent development of the frozen soil body;
- the average frozen soil temperature to verify the time-dependent frozen soil parameter (e.g., strength, stiffness, etc.) used for the structural design;
- the time required to freeze the designed frozen soil body;
- optimization of the freeze pipe arrangement and freezing operation;
- the time-dependent temperature distribution in the soil;
- determination of energy consumption; and,
- design of the capacity of the refrigeration plant (for brine freezing).

Figure 10 shows the temperature distribution in a frozen soil body after a certain freezing time from a thermal FEM analysis.

Structural Design. Structural design is required when the frozen soil body serves both as structural element and to cut off water. For many practical applications, it is sufficient to check the stress in the structural design of the freeze walls. The required structural design data for the allowable stress — Young's modulus and shear parameters — will be determined using the projected frozen soil stand-up time.

The basis for the sound structural design of a load-bearing frozen soil structure is extensive knowledge of the time- and temperature-dependent strength and deformation properties of the material. Frequently, the complex time-dependent stress-strain characteristics are simplified for the structural design. However, in some cases, the entire stress-strain-history from the start of loading has to be considered. Detailed time-dependent solutions can be reached by using numerical methods. Strength properties of the frozen soil body change according to the temperature distribution throughout its cross section. Whereas the strength is highest in the center of the frozen soil body, strength decreases towards its boundaries (0 degree Centigrade [32 degrees Fahrenheit] isotherm for fresh water). Due to this fact, it is possible that the stress at the boundaries exceeds the load support capacity of the soil body. In this case, plastification and subsequent stress redistribution will take place. The resulting stress distribution is now similar to that of the temperature distribution.

Final structural analysis of the freeze wall can be performed using FEM. Frozen and unfrozen soil stress-strain behavior can be simulated with a non-linear model. The advantage of using FEM is that it realistically accounts for both the frozen and the unfrozen soil. A major disadvantage of a more direct analytical approach is that it uncouples the frozen soil wall from the surrounding unfrozen soil and the external loads of earth and water pressure (plus any surcharge loads that are applied to the freeze wall).

The calculated deformations should include the effects of:

- excavation (stress relief and redistribution); and,
- creep characteristics of the frozen soil thawing.

Depending on the wall shape (straight or curved), the frozen soil body may only take limited loads; therefore, the design of additional support measures is required. These additional measures may consist of shotcrete application, or support measures such as anchors, beams or similar structures.

Design of Freeze Plant Capacity. The required capacity of the freeze plant (brine freezing) or the required amount of liquid nitrogen depends, among other things, on the number and length of freeze pipes, the total volume of the frozen soil, the freeze time and the required freezing temperature. In addition, the freeze plant must also be integrated into the construction program. A freeze plant may consist of several freeze units that are connected to one system based on the required capacity. After the freeze wall is built up to its required shape and thickness, it is usual practice to provide an intermittent level of cooling that is needed for the maintenance of the

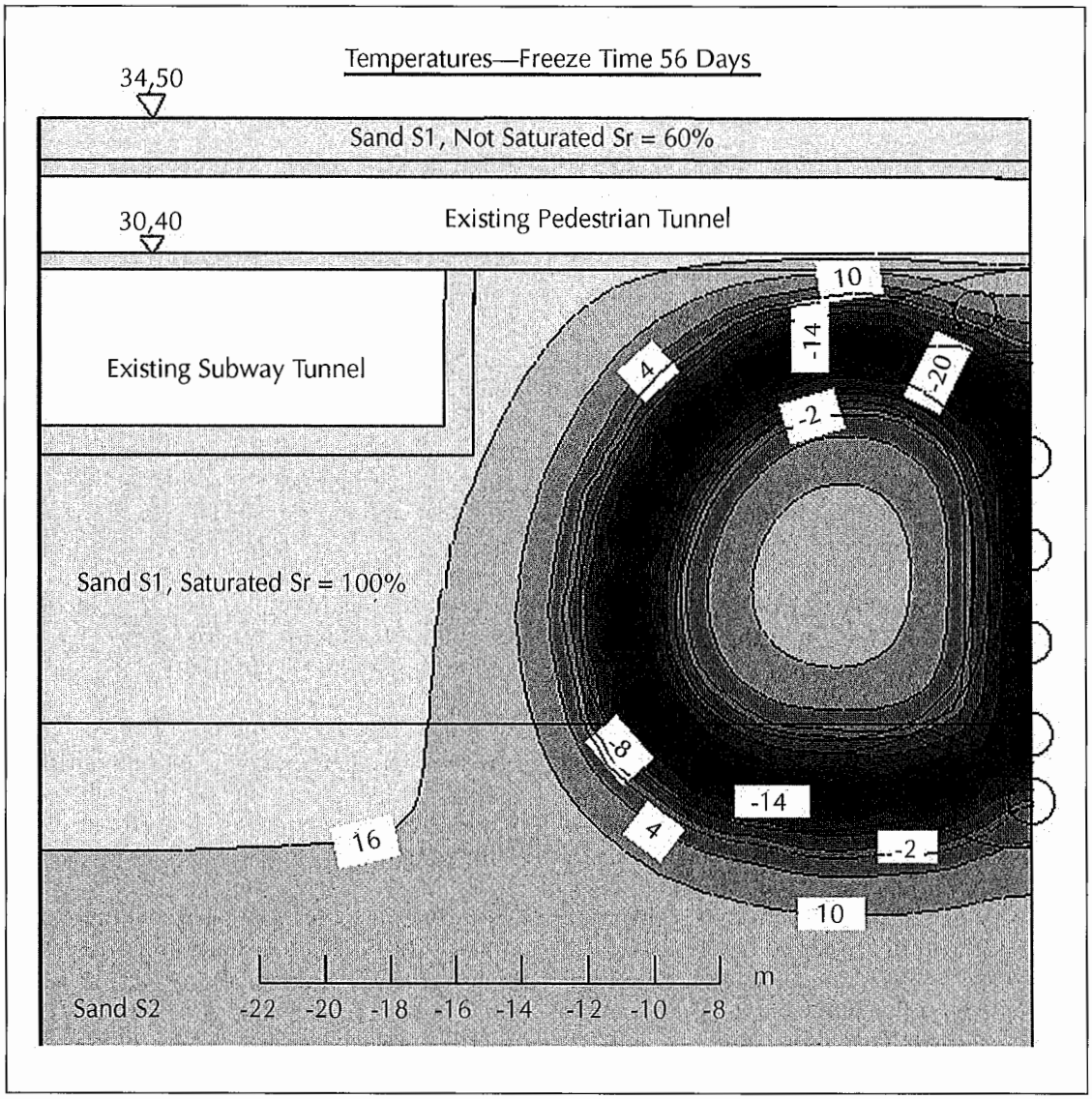


FIGURE 10. Results of a thermal FEM analysis.

frozen wall. Its capacity and its time-dependent development will be based on the thermal design analysis (as discussed above).

Risk Analysis. A sufficient risk analysis and appropriate measures to deal with these risks should be provided. Potential risks are:

- deviation of freeze pipe drillings;
- areas with unexpected high groundwater velocity;
- high groundwater salinity or contamination;
- inexperienced contractor;
- unexpected leakages;
- frost heave deformations;
- freeze plant/unit failure; and,
- electricity power failure.

Monitoring & Quality Control During Conduction

The construction of the frozen wall has to be monitored. Aside from measuring the temperature of the cooling agent, soil temperature measurements are of utmost importance. All essential data should be collected in one place and analyzed with computer-supported methods.

The following measurements should be taken to check the operation of the freeze system:

- temperature at the inlet and outlet of the freeze plant (brine) or the tank (LN_2 outlet), as well as flow and return temperatures of the brine;
- pressure of the cooling agent at the pressure side of the pumps and at the collecting main (brine freezing);
- the flow volume of the brine in particular locations;
- the liquid level in the brine collecting tank;
- the density of the cooling agent; and,
- the brine (or nitrogen exhaust) temperature should be monitored at every single return (exhaust) line, checking the proper cooling agent flow.

To check and control the freezing process, temperature measurements in the soil are strongly required. The spacing of thermocouples in temperature holes should be adjusted to the expected critical areas. In the area of the expected conditions (critical layers, freeze wall connection to structures, etc.), the thermocouple or monitoring hole spacing should be reduced. Figure 11 presents the time-dependent development of the frozen soil thickness based on temperature monitoring data that is compared with results based on thermal FE-calculations.

It is important to survey the actual location and deviation of each and every freeze pipe and temperature monitoring pipe. The temperature readings for monitoring can only be evaluated sufficiently if the exact location of each thermocouple and their distances from the two closest freeze pipes are known, because high temperature gradients may occur in some areas of the frozen soil. Otherwise, it is not possible to properly evaluate the temperature data and to obtain the right assessments.

Based on the survey data of all the pipes, the actual location of the pipes can be evaluated to determine if the freeze pipe spacing is sufficient at all locations. If required, additional drillings can be conducted before the freezing operation starts. The actual location of the freeze pipes and the thermocouples can be incorporated in the thermal FE model and the predicted time-dependent temperatures can be used for direct comparison with the related thermocouple readings during the freezing operation.

To guarantee the tightness of the freeze pipes, pressurization tests inside every pipe are strongly recommended before the freeze supply lines are connected. If freeze pipe leakages occur, brine would penetrate into the soil and would cause the immediate thawing of the frozen soil in that area. Depending on the amount of brine that has leaked, serious problems with the freeze wall can occur.

The monitoring program should also measure for deformation. These measurements should include:

- frost heave measurements of existing buildings or structures and the ground surface;
- inclinometer, or similar measurements, in the structural frozen soil body; and,
- extensometer measurements to check frost heave deformations in the subsurface.

There should also be pore pressure monitoring using pressure transducers, as well as groundwater level monitoring.

Completed Projects

Subway Section 3.4H, Düsseldorf, Germany. As part of the Düsseldorf mass transit subway system expansion, four 40-meter (128-foot) long tunnels were excavated directly below buildings and a major roadway. All four tunnels were advanced using SEM (formerly called the New Austrian Tunneling Method [NATM]). In each case, there was very little space between the roof of the tunnel and the bottom of the overlying building foundations. As a result, any ground loss, or other causes of settlement due to tunneling, would have led to direct and adverse movement of the existing building foundations.

The individual tunnels were located in granular soils. The general soil profile consisted of intermittent changing Quaternary sand

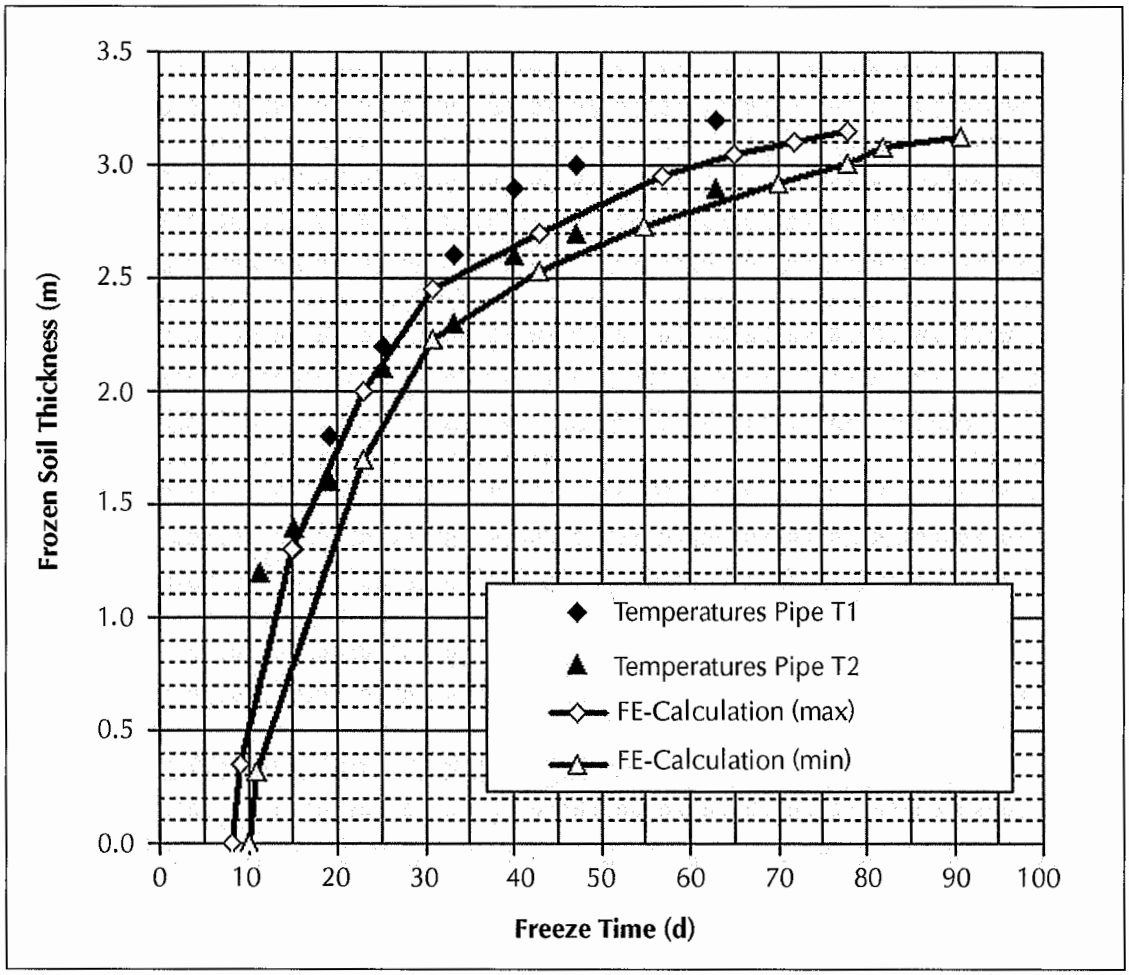


FIGURE 11. A comparison of temperature monitoring and FE-calculation results from the Westerscheldetunnel in the Netherlands.

and gravel layers. Underlying this stratum was very dense Tertiary fine sand.

For the driving of three of the tunnels, the gravel and sands were stabilized and the groundwater was controlled by ground freezing using a brine coolant (see Figure 12). Table 1 lists the data of the three frozen soil bodies for Tracks 1, 3 and 4.

For the ground freezing operation of the three tunnels, a freeze plant of two units with an operating brine temperature of –35 degrees Centigrade (–31 degrees Fahrenheit) and a capacity of 330 kW each were used. The freeze plant was installed in an isolated hall because of the urban residential area.

A unique feature of the ground stabilization for this project involved the freezing of natural unsaturated soil above the groundwater table. To ensure that the soil mass to be frozen had an adequate bearing capacity, water was injected into the soil. To do this, vertical cut-off walls were grouted along the sides of the tunnel to reduce the run-off of water injected into the soil. Water injection and freezing were conducted in four phases (see Figure 13).

All of the tunnels were driven without incident and with only negligible subsidence to the buildings and the main road directly above the tunnels. Figure 14 shows the very close distance from the buildings to the tunnel of Track 1 and the highly demanding urban conditions. The access shaft was located in the backyard of two residential buildings.

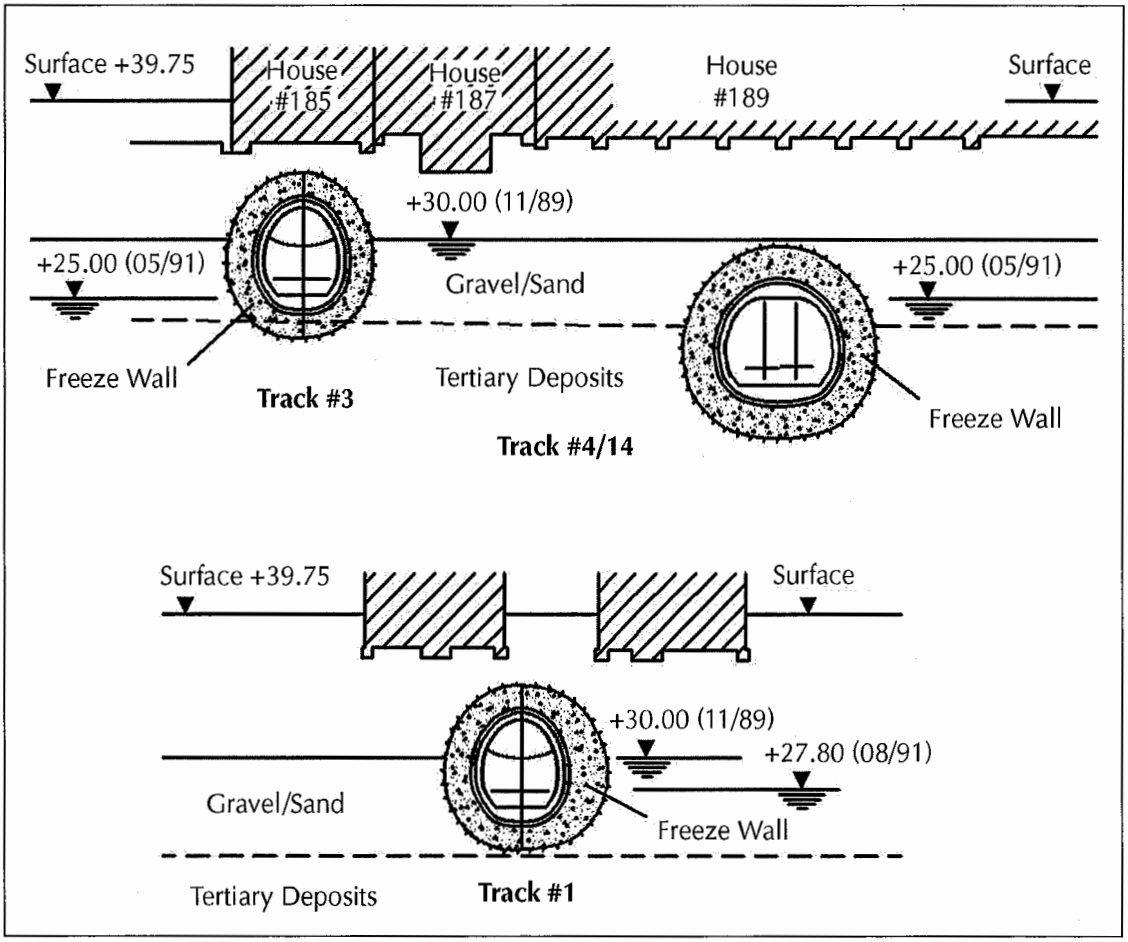


FIGURE 12. Cross-sections of the three track tunnels on the Düsseldorf subway project where brine ground freezing methods were applied.

Fahrlachtunnel in Mannheim, Germany. To relieve urban traffic congestion, the city of Mannheim in Germany built the Southern Expressway. The alignment required the construction of a 305-meter (978-foot) long tunnel that passes beneath a wide rail corridor with eleven frequently used railroad tracks in downtown Mannheim. During the entire con-

TABLE 1.
Data for the Düsseldorf Subway Project

	Top of Frozen Soul Below Foundation, m (ft)	Excavation Zone, m^2 (ft^2)	Frozen Soil Length, m (ft)	Frozen Soil Volume, m^3 (ft^3)	Frozen Soil Thickness, m (ft)
Track 1	0.6 (2)	46 (495)	48 (154)	1,600 (56,503)	1.5 (5)
Track 3	1.8 (6)	42 (452)	40 (128)	2,600 (91,818)	1.5 (5)
Track 4	6.7 (21.5)	75 (807)	40 (128)	2,900 (102,412)	2.2 (7)

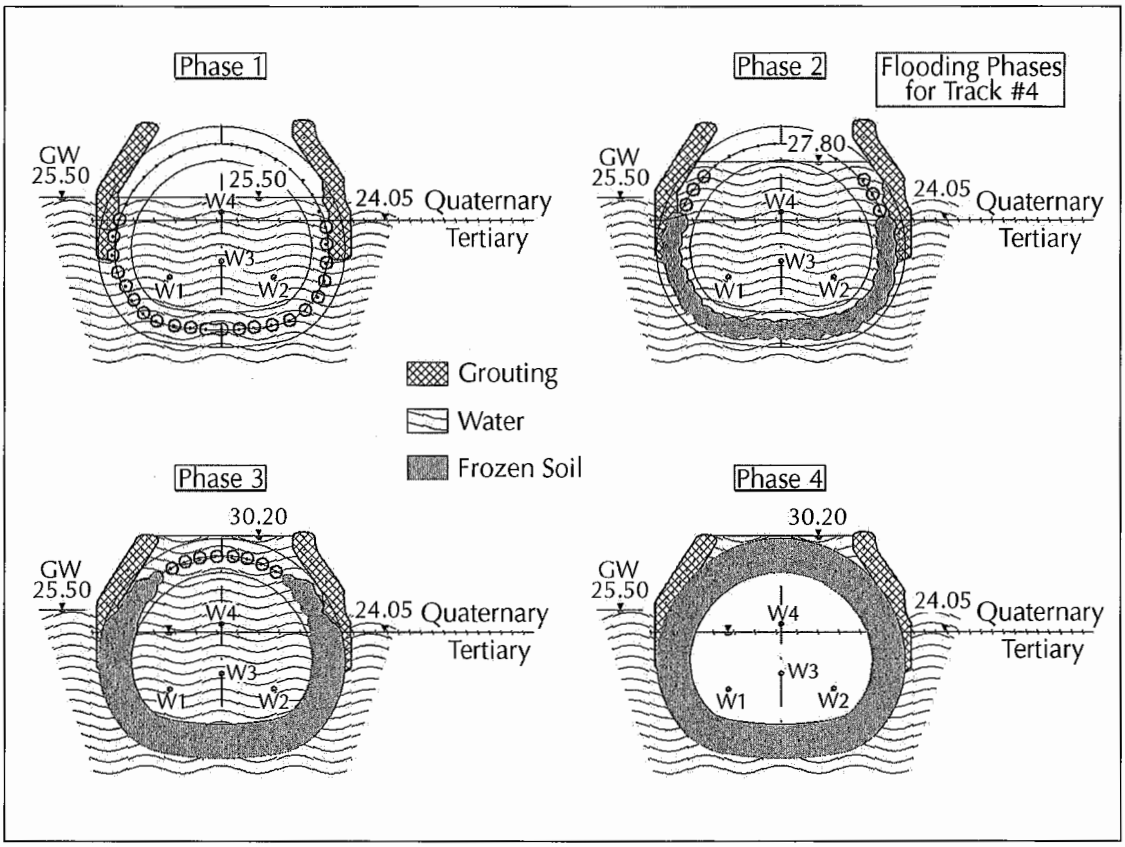


FIGURE 13. Water injection and freezing phases for the Düsseldorf subway project.

struction time no interruptions to the high-speed railroad service were to be tolerated.

The tunnel was divided in two cut-and-cover sections and one 184-meter (590-foot) long section directly underneath the railroad tracks. That section was advanced using SEM under the temporary protection of a structural and watertight frozen soil ring. The construction method was based on an alternative approach (value engineering) submitted by the contractor (joint venture). Figure 15 shows a model of the tunnel alignment and the railroad tracks.

The temporary frozen tunnel was located in very heterogeneous soil. The general soil profile consisted of a 4-meter (13-foot) fill layer followed by a 2-meter (6.5-foot) silt layer. Below the clay were alternating layers of sand and gravel with thicknesses ranging from a few centimeters to several meters. There was also a wide range of grain size distributions. The groundwater level ranged from 3.5 to 7.5 meters (11 to 24 feet) below the surface. A schematic longitudinal section with the geological profile is shown in Figure 16.

The top of the frozen tunnel was very close to the railroad tracks and the frozen soil body was partly above the groundwater level. The water saturation in this area had to be artificially increased to achieve the required strength of the frozen soil. An additive was administered to the water in order to increase its viscosity, thereby slowing the flow of the artificial watering. In addition, grouting measures were conducted to reduce the groundwater velocity in layers with high permeability.

The frozen tunnel length was divided by a transverse frozen soil bulkhead across its length. The width was also divided so that, in total, four tunnel-drifts, each 90 meters (289 feet) long, were advanced by using SEM. Figure 17 shows a cross-section of the tunnel using ground freezing.

An auxiliary tunnel with a diameter of approximately 2.4 meters (8 feet) was

FIGURE 14. Access shaft of the tunnel of Track 1 on the Düsseldorf subway project.

advanced above the groundwater level using SEM in order to install the freeze pipes for the transverse bulkhead as well as the pipes for water injection and grouting.

The drilling for the installation of the longitudinal freeze pipes started with conventional small-diameter drilling techniques from both sides towards the frozen soil bulkhead. The deviations of these shafts were so great that the required accuracy could not be achieved over the entire drilling length of 90 meters (289 feet). The accuracy was only acceptable for lengths between 40 to 50 meters (128 to 160 feet), which was not sufficient for the project. To solve this problem, microtunneling was required. The microtunneling technique used to install the long freeze pipes utilized a diameter of 47 centimeters (18.5 inches) to obtain the required accuracy. Overall, 86 microtunnels were drilled. Each tube contained two freeze pipes. After the installation of the 90-meter (289-foot) long freeze pipes, the remaining space inside the microtunnel tubes was filled with a cement mortar. The summary parameters for this ground freezing project are:

- There were four frozen tunnel drives;
- The depth of the frozen soil top below the railroad varied from 3.7 to –7.0 meters (12 to –22.5 feet);
- The excavation zone for each tunnel drive was 100 square meters (120 square yards);
- There were 86 microtunnels, each with a

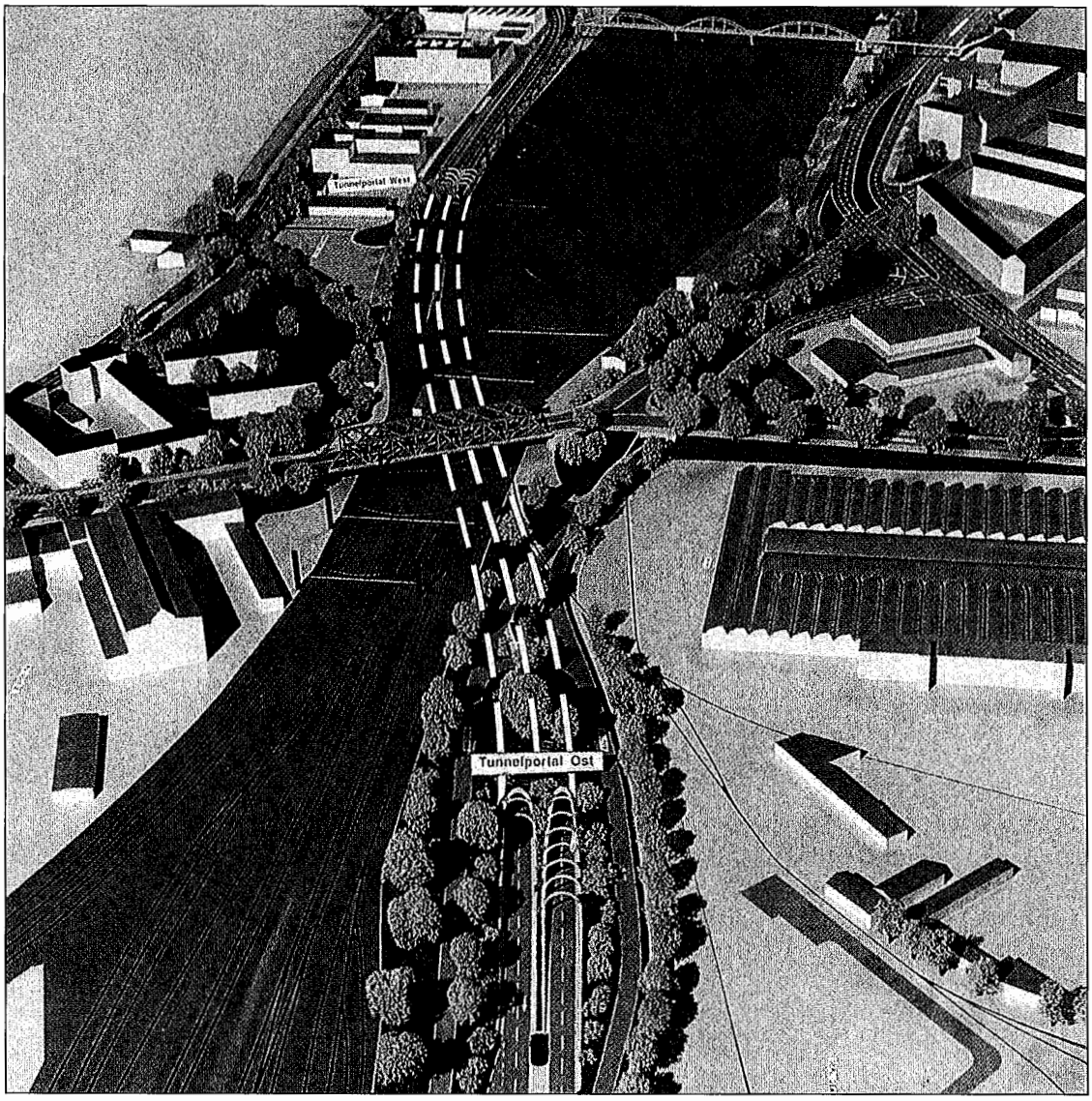


FIGURE 15. A model of the tunnel and railroad alignment for the Fahrlachtunnel project.

47 centimeter (18.5 inch) diameter, for a total length of 7,900 meters (4.8 miles);

- Total frozen soil length was 184 meters (590 feet);
- Total frozen soil volume was 27,000 cubic meters (35,313 cubic yards);
- Frozen soil thickness was 1.75 meters (5.5 feet), with a saturation of 1.0;
- Average frozen soil temperature was –10 to –12.5 degrees Centigrade (14 to 9.5 degrees Fahrenheit);
- Freezing time to closure was 30 days;
- Initial freezing time (time to freeze the soil to the required thickness) was five to eight weeks;
- Frozen ground maintenance freezing time was approximately more than four hours per day; and,
- Temperature monitoring was conducted with approximately one thermocouple per cubic meter of frozen ground, for a spacing of either 0.5 or 3.0 meters (1.5 to 9.5 feet).

For the ground freezing operation for the four tunnel drives, there was a freeze plant that

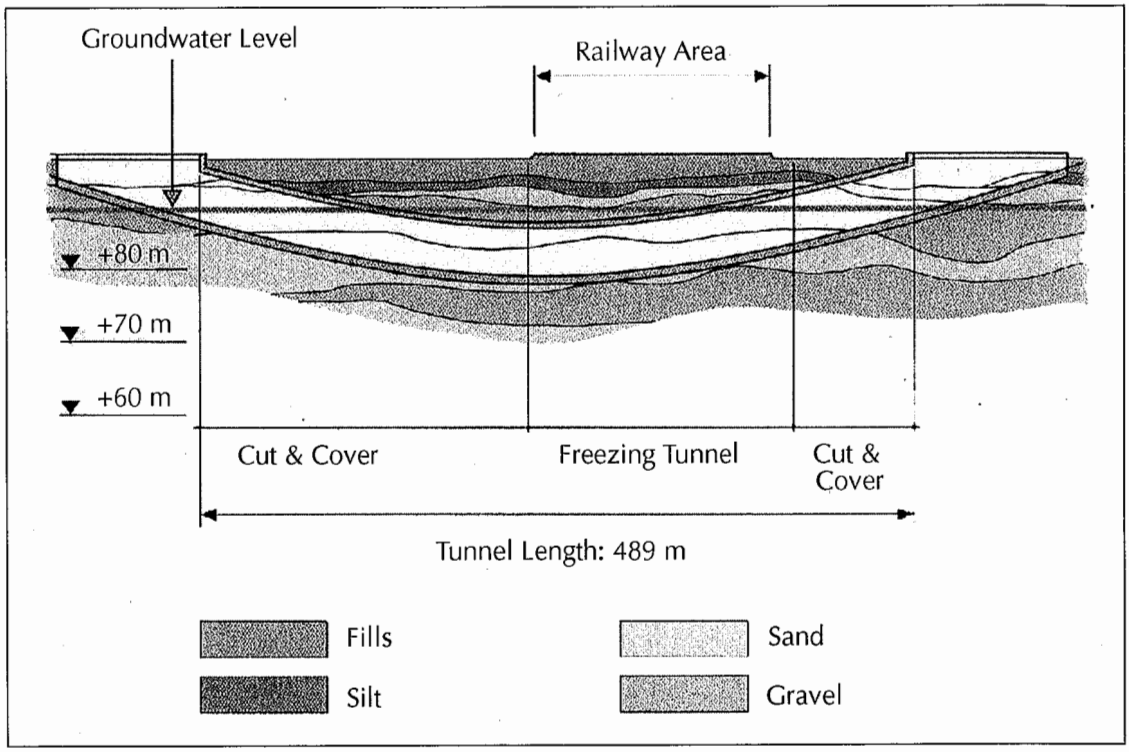


FIGURE 16. Longitudinal section of geology for the Fahrlachtunnel project.

consisted of four independent units. Operating brine temperature of the plant was –35 degrees Centigrade (–31 degrees Fahrenheit) and the plant had a total capacity of 1,680 kW. The plant was designed for the initial freezing of one 90-meter (289-foot) long tunnel drive and for the frozen ground maintenance of a second 90-meter (289-foot) tunnel drive at the same time.

During excavation, an initial shotcrete lining with a thickness of 35 centimeters (14 inches) was applied. The final concrete lining, with a thickness of 50 centimeters (20 inches), was installed in each tunnel drive after the initial lining was placed.

So far, the Fahrlachtunnel is the largest tunnel construction project that has used ground freezing worldwide. Figure 18 presents a view to the launching shaft at the south side of the tunnel.

Rehabilitation of a Jet Grouted Seal Block for Fernbahntunnel, Lot 3, in Berlin, Germany. The Fernbahntunnel Project Lot 3 consisted of construction of an underground railway with four tracks. For the tracks, four single tubes with an excavation diameter of 8.9 meters (28.5 feet) and a 7.8-meter (25-foot) inside diameter were driven using slurry shield tunnel boring machines (TBMs). The launching shaft had a length of 40 meters (128 feet), width of 60

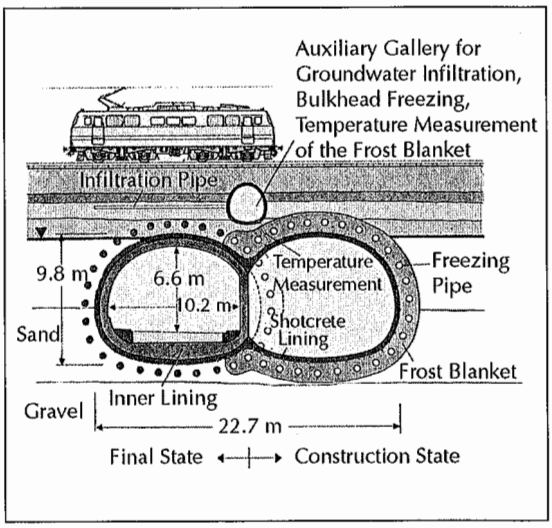


FIGURE 17. A cross-section of the Fahrlachtunnel.

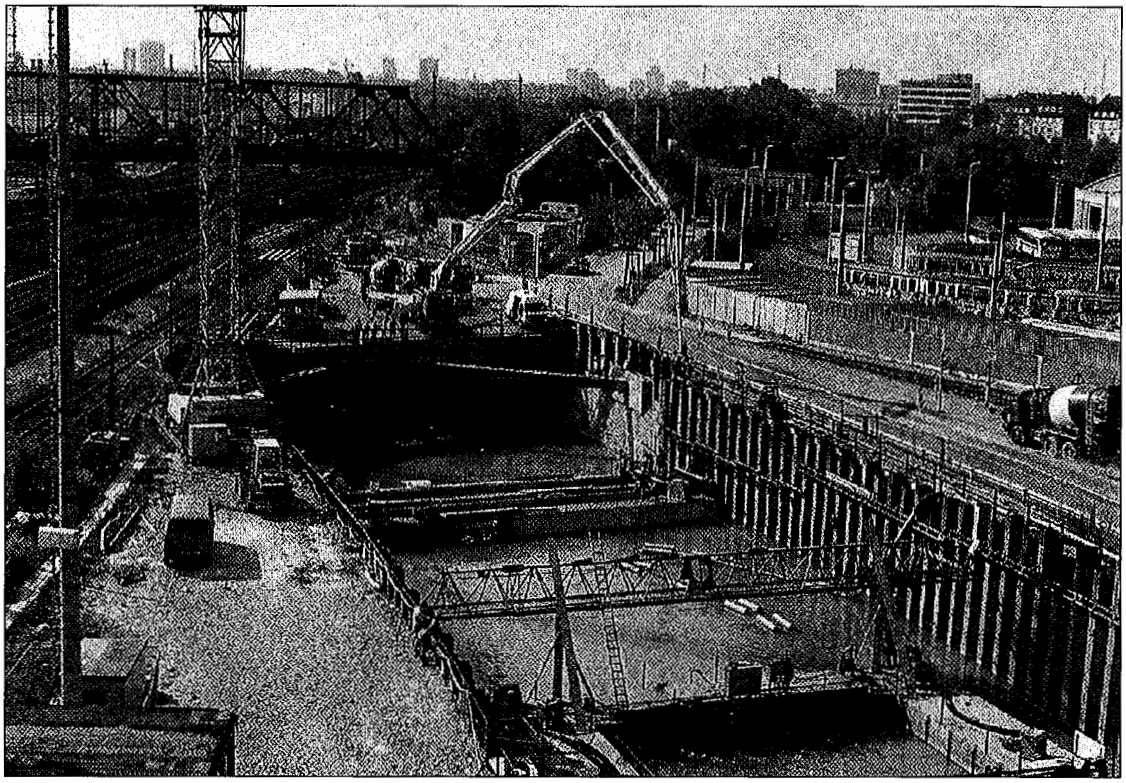


FIGURE 18. The southern side of the deep excavation for the Fahrlachtunnel.

meters (192.5 feet) and a depth of 15 meters (48 feet). It was constructed using a large caisson, which contained four openings that were temporarily plugged with non-reinforced concrete for the later launching of the four TBMs through the caisson wall. It was planned to remove the non-reinforced concrete prior to TBM installation. A jet grouted seal block was constructed after the caisson was sunk to seal the openings after the removal of the concrete. The seal block served both as a structural element (soil and water pressure) and to cut off the water.

During the removal of the concrete for the first opening, initially just water and then later large amounts of water and soil flowed into the caisson. The leakage could not be stopped and the workers had to be evacuated. The loss of soil outside of the caisson caused a sinkhole at the ground surface and the caisson finally was flooded in order to prevent more damage.

Because of the significant leakage of the seal block, and also due to the soil movement, numerous cracks in the seal block were assumed to have occurred. The decision was made to seal and strengthen the block using ground freezing. Ground freezing was selected because all existing cracks and leaks could be reached as the frost would grow in the whole block. Figure 19 shows a vertical section of the caisson and the seal block.

The undisturbed ground in the area of the caisson consisted of fine to medium sand with interbedded gravel layers a few centimeters thick over the whole depth. The groundwater level was approximately 3 meters (9.5 feet) below the surface. The existing seepage velocities were 1.5 meters (5 feet) per day and, therefore, not considered critical for ground freezing purposes.

For the remediation efforts, the thickness and the width of the seal block were enlarged with jet grouting techniques. The final total thickness was approximately 7.4 meters (24 feet) for structural and safety reasons.

The purpose of rehabilitating the seal block using ground freezing was to provide temporary groundwater cut-off and ground support

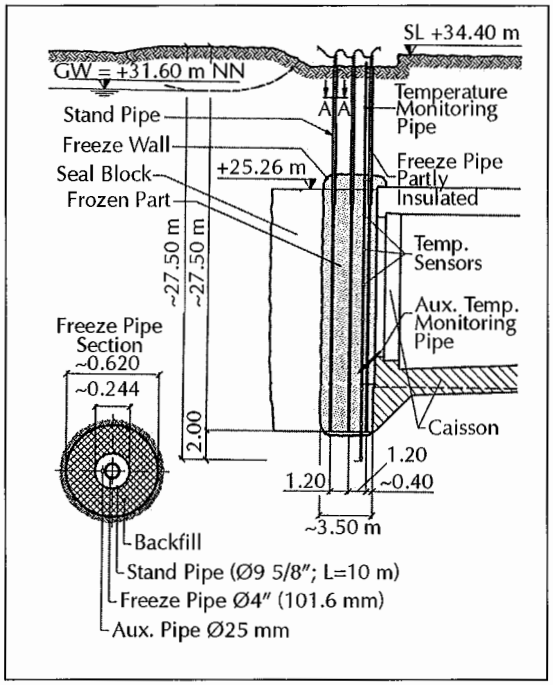


FIGURE 19. A vertical section of caisson and seal block for Fernbahntunnel Lot 3.

for the time that was needed to lower the water inside the caisson, remove the non-reinforced concrete plug and install a watertight membrane structure in the opening where the plug had been.

Based on the project's structural design, the required freeze wall thickness was 3.5 meters (11 feet). The freeze wall was width was 60 meters (192.5 feet) and its height was 16 meters (51 feet). It had a total frozen volume of 3,400 cubic meters (4,447 cubic yards). Based on thermal FE calculations, three rows of freeze pipes with the following spacings were chosen:

- Row 1 (next to the caisson): 0.9 meters (3 feet); and,
- Rows 2 & 3: 1.2 meters (4 feet).

The freeze pipe layout was required to meet the following demanding requirements for the freeze wall:

- Freeze wall temperature of ≤ –10 degrees Centigrade (14 degrees Fahrenheit);
- Outer edge temperature of ≤ –2 degrees Centigrade (28 degrees Fahrenheit); and,

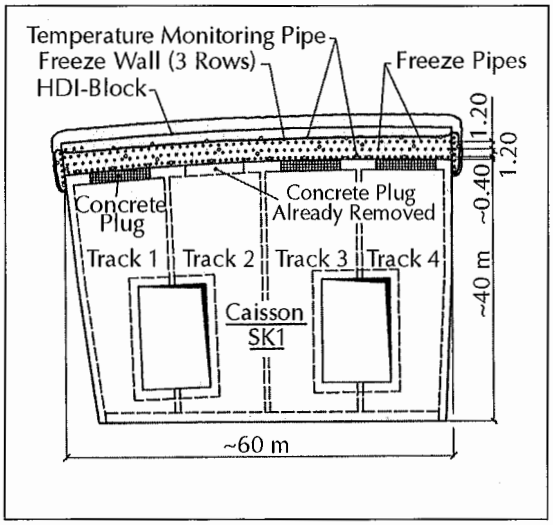


FIGURE 20. Top view and freeze pipe layout for Fernbahntunnel Lot 3.

- Temperature at the caisson interface of more than –10 and less than –20 degrees Centigrade (14 and –4 degrees Fahrenheit).

The last requirement was necessary to limit the temperature-induced stresses in the concrete of the caisson wall. Figure 20 shows a top view of the caisson with the freeze pipe layout.

Following are the parameters for the freeze system used for this project:

- Total of 184 freeze pipes in three rows — 66 in Row 1 and 50 each in Rows 2 and 3 (see Figure 21);
- Seven freeze pipes at the side;
- 35 temperature monitoring pipes;
- Six to ten thermocouples per monitoring pipe;
- A total of 350 thermocouples to monitor the ground and brine temperatures;
- Total freeze plant capacity of 465 KW at –35 degrees Centigrade (–31 degrees Fahrenheit);
- Cooling agent was $CaCl_2$ brine (30 percent);
- Freeze-up time for first section (Openings 1 and 2) to meet all design requirements was approximately five weeks, although an additional two weeks of freezing was required by the owner;

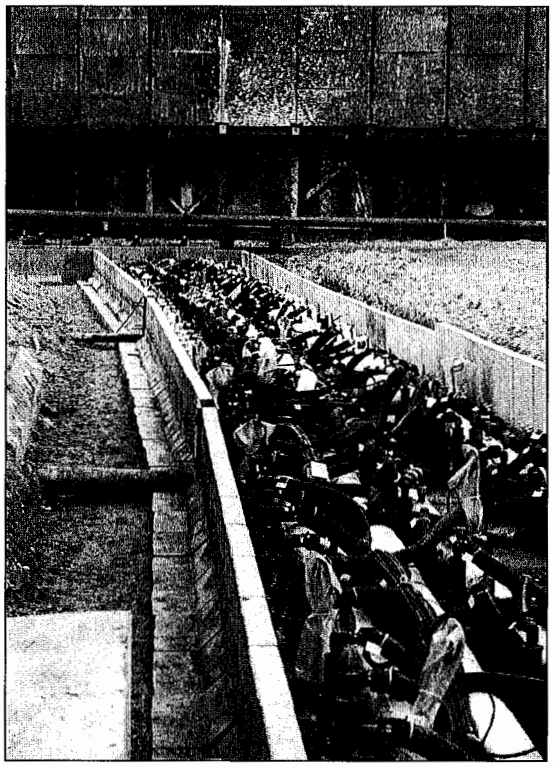


FIGURE 21. Freeze pipe header, supply and return lines on Fernbahntunnel Lot 3.

- Freeze-up time for second section (Openings 3 and 4) was about four and a half weeks (time was reduced due to pre-cooling from the adjacent section); and,
- Freeze wall maintenance phase in both sections was about six to ten hours of freeze time per day.

After removing the concrete plug, a special watertight membrane structure (developed by the main contractor) was installed under the protection of the freeze wall at each opening. This structure consisted of a circular steel ring that was attached to the edge of the opening, and a rubber membrane that spanned over the entire opening. Bentonite mud filled the space between the membrane and the frozen ground. Once a steady pressure on the mud was created, the freezing operation was turned off for that opening. Figure 22 shows the freeze wall after the removal of the concrete plug and prior to membrane installation. The membrane could be dismantled and removed after the TBM was installed in the launch ring and the chamber in front of the cutting wheel was pressurized by compressed air.

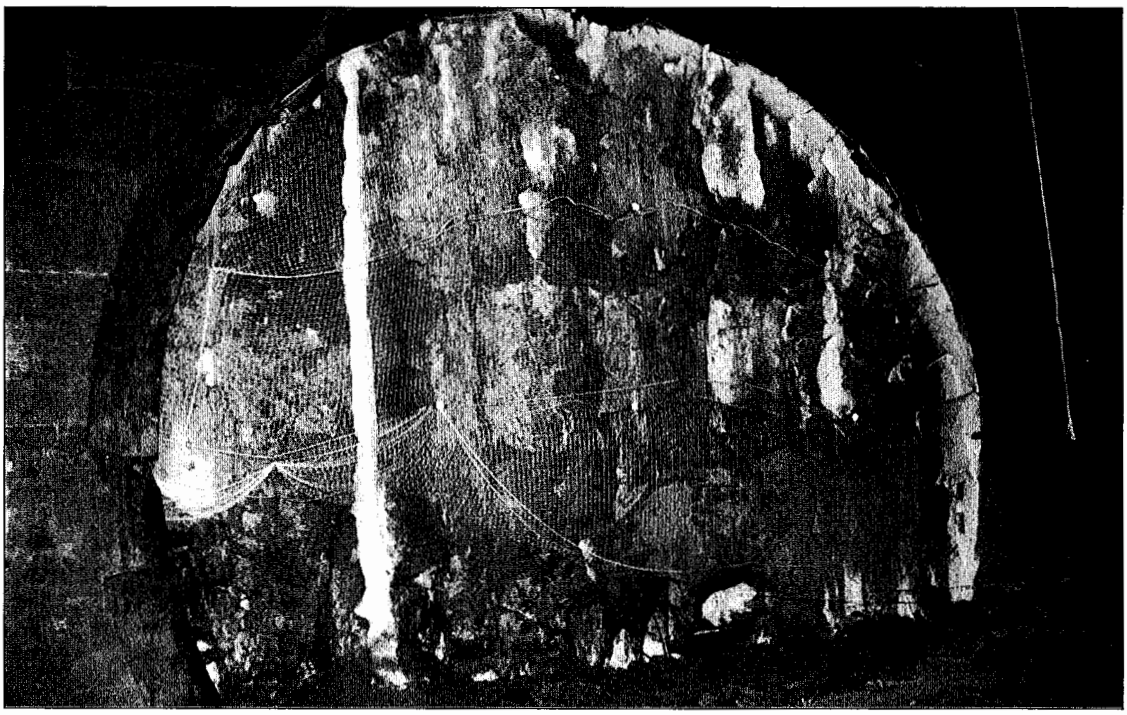


FIGURE 22. Freeze wall exposed after the removal of the concrete plug.

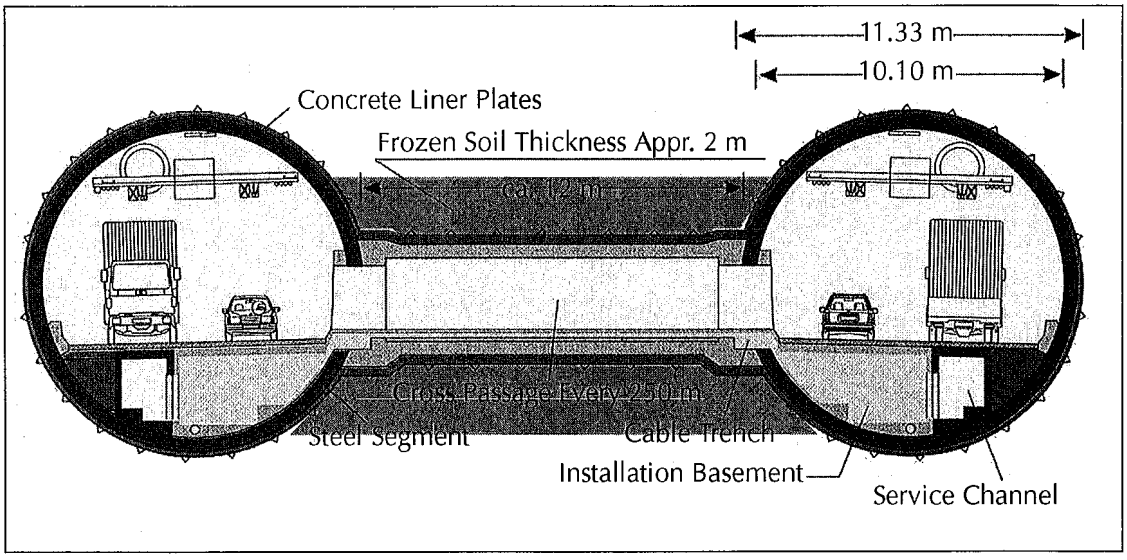


FIGURE 23. Section in the area of a cross-passage for the Westerscheldetunnel.

Prior to launching the air-pressurized hydro-shield TBMs, all freeze pipes had to be removed and the holes had to be backfilled with grout in order to prevent the sudden loss of pressure in the chamber in front of the cutting wheel during drive of the TBM through the seal block. Therefore, the freeze pipes were heated until the ground temperature was approximately +2 degrees Centigrade (35 degrees Fahrenheit).

All line connections were changed from the freeze plant to a heat plant, which consisted of two electrical immersion type heaters with a total capacity of 150 kW. The heated brine temperature was +30 degrees Centigrade (86 degrees Fahrenheit). It took approximately 33 days to heat up the ground as required. In the end, all pipes were extracted except one, which was removed by being overdrilled.

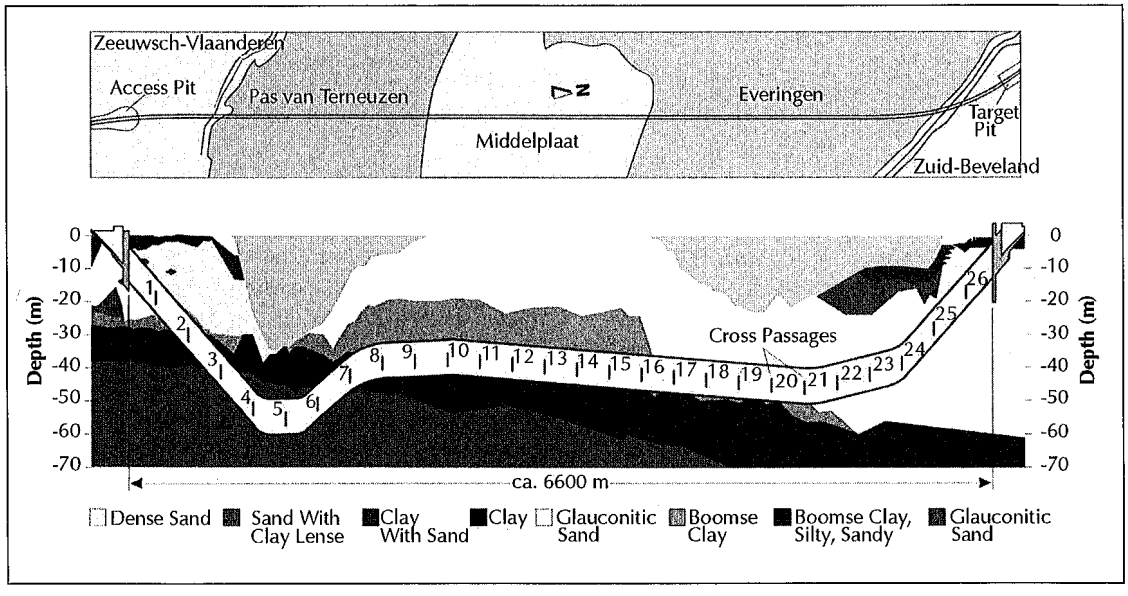


FIGURE 24. Longitudinal section and geology for the Westerscheldetunnel project.

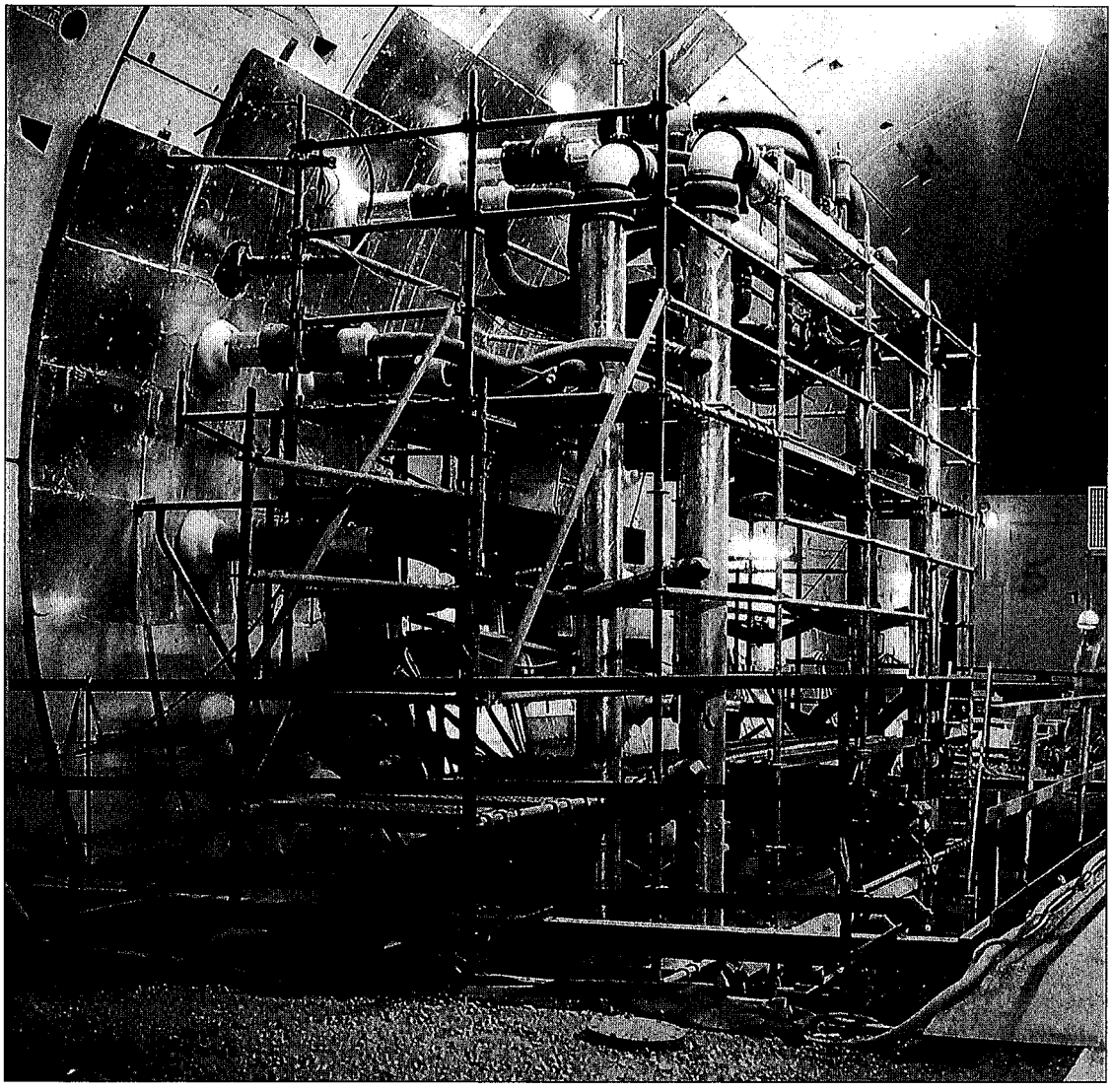


FIGURE 25. Main tunnel with installed freeze pipes for the cross-passage for the Westerscheldetunnel.

Westerscheldetunnel, Terneuzen, the Netherlands. Located in the western part of the Netherlands, the Westerscheldetunnel connects Zeeusch-Vlaanderen with Zuid Beveland on the continent. The tunnel project consisted of two tubes, with each tube containing two road lanes. The tunnel tubes were driven under the North Sea using two TBMs with a pressure-balanced hydro-shield. The bore diameter of the tunnels was 11.33 meters (36 feet). Each tunnel tube had a length of 6.6 kilometers (4 miles) and the lowest elevation was approximately 60 meters (192.5 feet) below mean sea level.

For safety requirements, the tunnels were connected by cross-passages every 250 meters (802 feet). A total of 26 passages with a clear cross-section of 6.25 square meters (7.5 square yards) and an average length of 12 meters (38.5 feet) were built (see Figure 23). The passages were constructed under the temporary protection of a watertight and structural frozen soil body. The excavation for the cross-passages was done using NATM.

The individual passages were located in both cohesive and non-cohesive soils. The general soil profile consisted of 20 to 30 meters

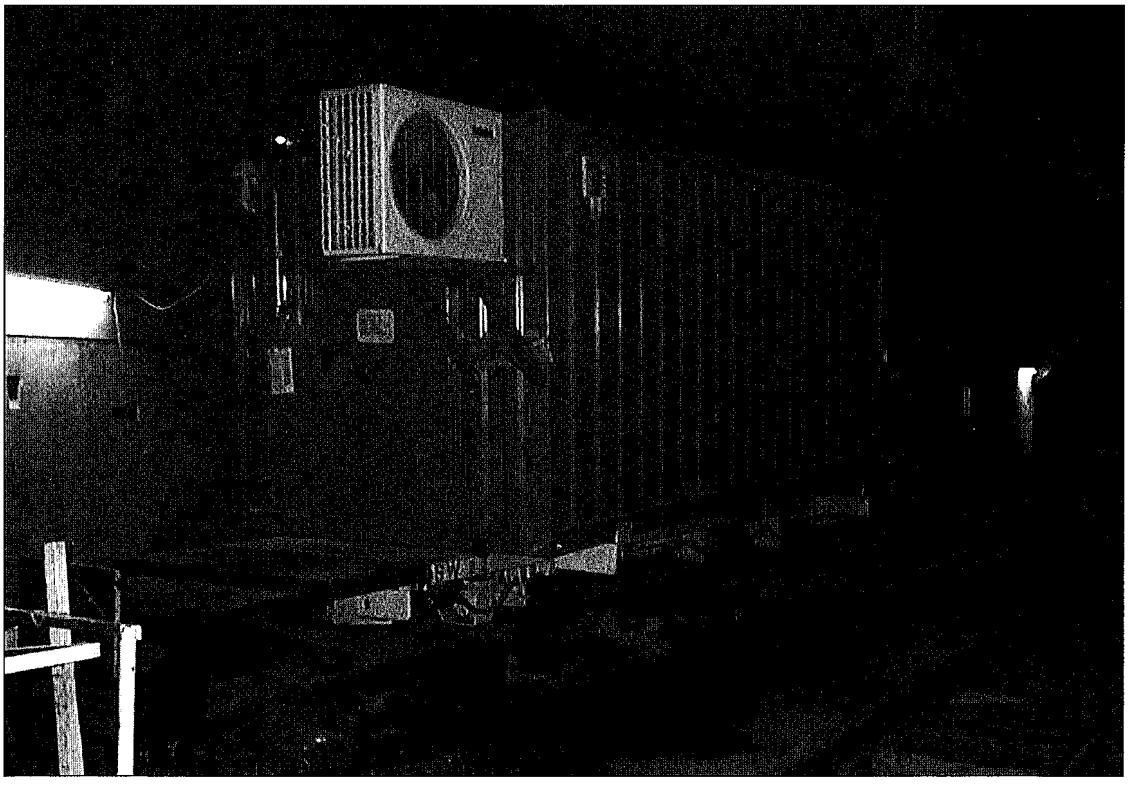


FIGURE 26. Freeze unit inside the Westerscheldetunnel.

(64 to 96 feet) of medium dense sand with embedded layers of clay, peat and sea silt. Underlying this stratum was stiff clay 8 to 28 meters (25 to 90 feet) thick. Below the clay was very dense sand that was hydraulically connected to the upper medium dense sand and the seawater level. As a result, water pressures of 6.5 bars at tunnel depth had to be considered for the design and construction (see Figure 24).

In addition to available soil data provided by the owner, further soil investigations and lab testing were conducted that focussed on determining the soil properties and behavior once the soil was frozen and after it had thawed.

Each cross-passage freeze zone consisted of 22 freeze pipes and two temperature monitoring pipes. The freeze pipes were located around the excavation line as shown in Figure 25.

A major construction problem that was encountered during the installation of the freeze pipes was extremely high water pressure. To accommodate the high pressure, a special sleeve was designed for the freeze pipes. The borings were drilled from the eastern tunnel tube to the western tube, using a specially enhanced lost bit drilling technique.

For the ground freezing operation of each cross-passage, a freeze unit with an operating brine temperature of –37 degrees Centigrade (–35 degrees Fahrenheit) and a capacity of 94 kW capacity was used. The freeze unit was located inside the launching tunnel tube directly beside the cross-passages in order to avoid long conduits that would cause heat losses (see Figure 26).

Difficulties in achieving complete closure of the frozen soil at the interface of the cross-passage and the receiving tunnel lining were recognized during the drilling of the first cross-passage. The problem occurred due to a modification in a part of the tunnel where steel lining segments were used. The steel lining at the cross-passage interface resulted in a larger heat transfer than was originally anticipated.

In the parts where the frozen soil body was connected to reinforced concrete lining seg-

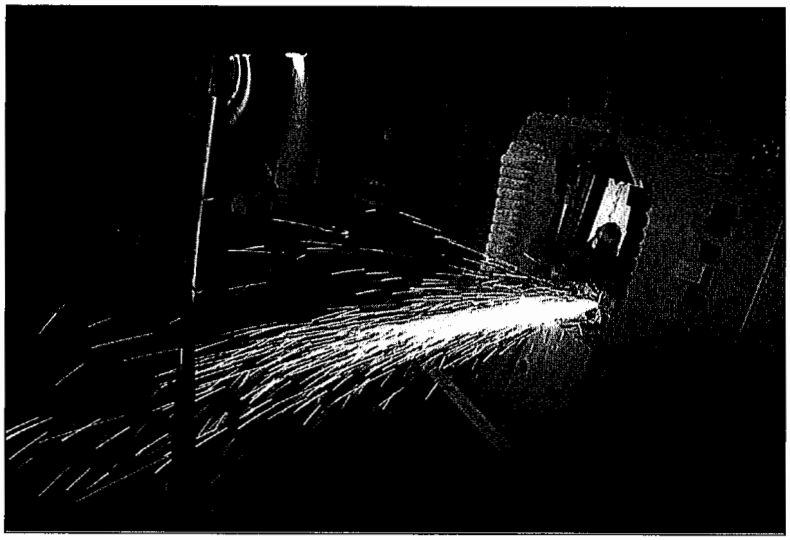


FIGURE 27. Cross-passage opening inside a Botlekspoortunnel tube.

ments no problems occurred. However, innovative methods to attain the proper freeze temperatures were needed. For the first cross-passage, dry-ice was used to cool down the steel lining elements at the receiving tunnel. Doing so resulted in an effective closure.

For the remaining cross-passages, a temporary curtain around the steel lining segments was installed. Inside the curtain room the air was cooled down using a special air-conditioning unit. Closure was achieved in every case without problem.

Botlekspoortunnel, Rotterdam, the Netherlands. The Botlekspoortunnel is part of the Betuweroute, a double-track freight train route, that links the harbor of Rotterdam with the European mainland. The project is considered one of the biggest infrastructure projects in the Netherlands. The Betuweroute is 160

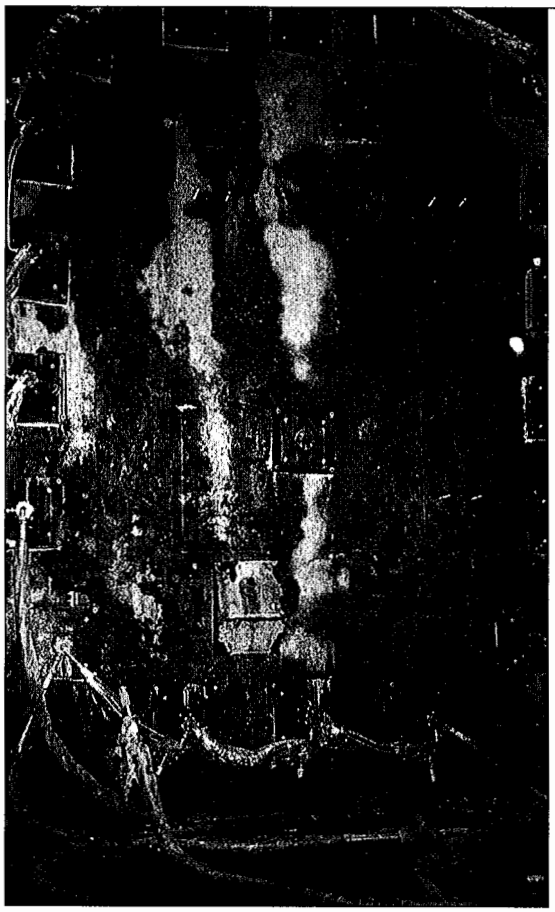


FIGURE 28. Freeze pipes inside the deep shaft for the Botlekspoortunnel.

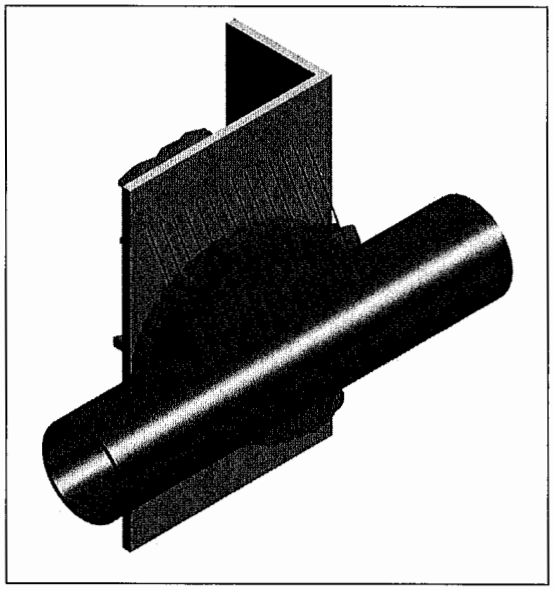


FIGURE 29. A model of the cross-passage between the shaft and tunnel tube for the North-South City Train Line, Cologne.

kilometers (97 miles) long and requires an investment of approximately 4 billion euros.

The Botlekspoortunnel is a double-track tunnel and is the first railway tunnel in the Netherlands that was built using a pressure-balanced hydro-shield. In total, it is 3,400 meters (2 miles) long — 1,850 meters (5,935 feet) were built by shield driving.

For safety requirements, the tunnel tubes had to be connected by three cross-passages in the area of Rotterdam. These cross-passages were also built using the ground freezing method. But, in contrast to the Westerscheldetunnel, an additional deep diaphragm concrete wall shaft was built in the middle between the tunnel tubes. Once that shaft was built, freeze pipe drilling was conducted from both sides of the shaft to the adjacent tunnel tube.

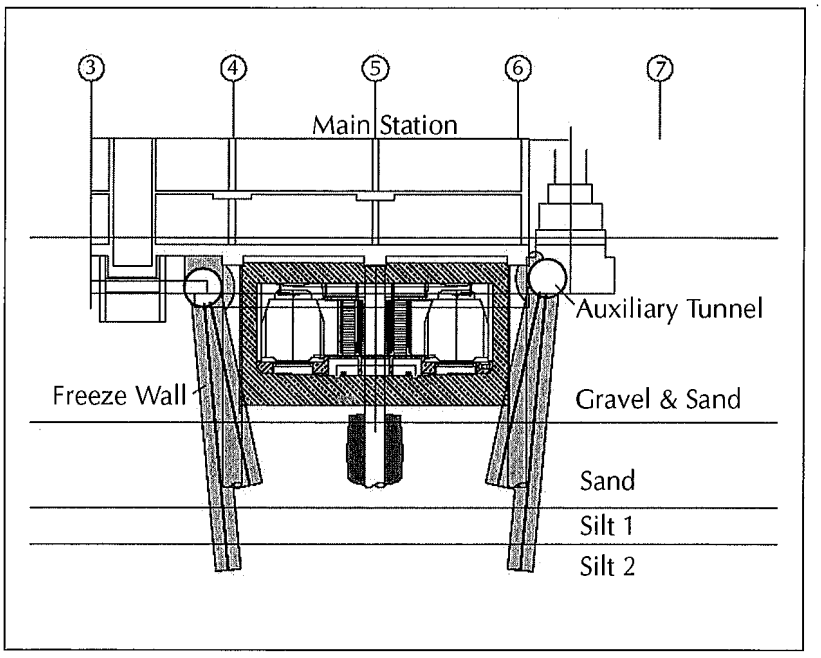


FIGURE 30. The deep excavation underneath a main station in Leipzig, Germany, using ground freezing.

Because of that shaft, the freeze pipes were very short; most had lengths of less than 2 meters (6.5 feet). Since the volume of the frozen soil was relatively small, the liquid nitrogen freezing method was chosen. The cross-passages were driven using the mining method, starting from the sinking shafts to both tunnel tubes.

The cross-passages were at a depth of approximately 25 meters (80 feet) below the ground surface and the groundwater level. The individual passages were mainly located in non-cohesive soils (medium to very dense sands). Only small parts of cohesive layers were present in the area of ground freezing.

The freeze-up of the frozen soil body to the required thickness of 1.5 meters (5 feet) took only one week due to the use of liquid nitrogen. Figure 27 shows the opening of the cross-passage inside a tunnel tube when the excavation from the shaft was finished. The frozen soil body served both as a structural element (soil and water pressure) and to cut off the water. Figure 28 shows the freeze pipe layout from inside the deep shaft.

Current Projects

The following projects that utilize the ground freezing method are currently under construction or are planned for construction, or have been recently finished:

- Marienplatz, Munich, Germany — The expansion of an existing underground subway station underneath the city hall consists of two freezing sections each 100 meters (320 feet) long. Currently finished.
- Subway Vienna, Austria — Undercrossing the Danube canal, length approximately 70 meters (225 feet). Freezing operation started in 2005.
- U5 Shuttle — Underground station Brandenburger Tor Subway line U5, in Berlin, Germany. Construction of an underground station using SEM under the protection of frozen soil. Freezing operation started at the end of 2005.

- North-South City Train Line, Cologne, Germany — Construction of one large tunnel section (switching station), one large section of an underground station and several cross-passages (see Figure 29). Freezing operations start 2006/2007.
- Randstad Rail Rotterdam, The Netherlands — Construction of five cross-passages between two TBM-driven tunnel tubes. Freezing operation starts 2006/2007.
- Hubertustunnel Den Haag, The Netherlands — Construction of five cross-passages between two TBM-driven tunnel tubes. Freezing operation starts 2006/2007.
- Airport-City Train, Hamburg, Germany — Construction of four cross-connections between two caissons and two TBM-driven tunnel tubes. Freezing operation started in 2005.
- Central Station, Rotterdam, the Netherlands — Large and deep excavation at a main station highly frequented by traffic and pedestrians, combining concrete diaphragm walls and freeze walls.
- City Tunnel, Leipzig, Germany — Undercutting of an old main station (see Figure 30) for the construction of a two-track city train line with excavation underneath an existing old main station 16 meters (51 feet) deep, using frozen retaining walls (structural and water cut-off). Length approximately 80 meters (257 feet). Currently being planned.

Summary

In many cases, ground freezing is the only solution to create a watertight and load-carrying soil body in water-bearing soil. However, numerous other applications are possible and cost-effective solutions can be realized even under highly demanding conditions. Ground freezing has become more popular and is going to be used more frequently in the future. Its use, in particular, in tight, conjested urban environments is increasing. Essentially, ground freezing is a very safe method but it has to be designed by engineers who are experienced with ground freezing and who use sound principles. In addition, it is important that the ground freezing operation must be monitored throughout the construction process to assure safety.

NOTE — *This article is based in part on a presentation given in March 2000 to the BSCES Geotechnical Group.*

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Moss on the Median

When looking at designing or updating major thoroughfares it might be good to look at more issues than just traffic flow and safety.

BRIAN BRENNER

Before Route 3 was widened north of Boston, it offered motorists the illusion of driving through the wilderness. The two northbound and southbound expressway lanes were separated and insulated from their surroundings by wide swaths of woods, both in the median and along the rights-of-way. For most of the ten or so miles from the Route 128 junction north to Interstate 495, the road seemed to be isolated in its forest and not a ride through suburbia. Come weekends in early October, the old highway was in its glory as the trees started to turn color. One could drive at a pleasant clip for miles under a dark blue autumn sky, with orange light sparkling off the shimmering maple trees. There was the promise of pumpkins and crisp Macintosh apples at the country farms along its exits.

This illusion of a drive through the woods was created, in part, by the highway's original method of design and construction. Instead of plowing over the full right-of-way and planting grass, the road was placed and built with comparatively little disruption to the surrounding terrain. This approach was particularly true for the median strip, which remained densely wooded.

Unfortunately, soon enough commuter traffic on Route 3 moved at more of a slow crawl through the woods than a pleasant drive. Traffic studies determined that it was time to widen the four-lane highway to six lanes and improve the substandard interchange layout. Once construction began on the widening project, one of the first things to go were the trees in the median. This space was deforested. Areas of deep gullies were filled in and paved over. The widened, reconstructed highway now looks much different. It is a well designed, vastly improved highway in terms of traffic flow. However, what used to be a drive through the woods has been replaced by a somewhat dreary, characterless slog — it has been turned into *Everyexpressway*. The forested median is now an open, grassy space. Whereas the old highway used to fit — in its way — into the landscape, the new version is a wide, open gash that seems to make as big an impact (and scar) on the terrain as possible.

Parkway vs. Expressway

Limited access highways are a relatively new type of infrastructure, dating back to the 1910s. Some of the earliest highways were "parkways" that were constructed in New York. These highways had architecturally sculpted viaducts (what we think of as "context-sensitive design" today), and they were

designed more for leisure travel than commuting. As car culture took hold and traffic volume increased, parkways gave way to "expressways." Expressways were the forerunners of the current Interstate highway program, which has standardized and institutionalized the form. Unlike the pastoral parkways, expressways were no-nonsense highways, intended to be fast, efficient, and built in part to provide transportation facilities for military use during wartime. It is only more recently that aesthetics and context-sensitive design (where the facilities are understood and even designed to be part of their surroundings) are considered. As part of these recent improvements, expressway bridges now have road and place name markers attached in an effort to help drivers feel more connected to the landscape through which they are driving.

Parkways became more widespread thanks to Robert Moses, the late commissioner of the New York Tri-Borough Bridge and Tunnel Authority. Under his leadership, parkways and many aspects of the suburban form were developed around New York City, particularly on Long Island.[1] Parkways were designed to be not just limited access highways, but an aesthetic experience. Therefore, rights-of-way featured trees and fields, and overpasses were adorned with stone veneer and architectural details. Parkways were not originally conceived as massive people movers, but were thought of as roads for pleasure drivers. Caro documents some other unsavory aspects of the original designs, such as the deliberately low overpasses intended to keep busses and mass-transit riding city dwellers away from using the parkways and, thus, invading suburbia.[1]

Modifying Parkways

Today, on Long Island, you can drive on two types of freeways: the typical, brutal, no-nonsense expressway like the Long Island Expressway, or one of many parkways such as the Northern Parkway. The parkways' original Sunday drive function has long been superseded by the hordes of commuters that motor back and forth each day. The Long Island parkways still have wooded rights-of-way, and the viaducts are still sheathed in attractive stone. But these freeways have been rebuilt over the years for the purpose of moving traffic, with the addition of lanes and more modern geometries at their interchanges.

Traveling north from New York City, drivers have an interesting choice in Connecticut between the Merritt Parkway and Interstate 95 (originally the Connecticut Turnpike). The Merritt Parkway is an historic highway with neo-classical and modernistic style bridges festooned with all sort of ornamentation. Except for the crushing traffic, a jaunt on Merritt Parkway is pleasant and invigorating, with long stretches of woods and a canopy of trees to drive through. Interstate 95, on the other hand, is a particularly ugly, placeless highway that takes no prisoners as it makes its Sherman's march across the landscape to New Haven.

With time, Interstate 95 has softened, and the Merritt Parkway has hardened. Interstate 95 has undergone reconstruction and landscaping improvements so that it is not as much of a scar as it used to be. Much of the reconstruction has been nicely done, with aesthetically shaped, attractive viaducts. On the other hand, the Merritt Parkway has been subject over the years to many ill-advised reconstruction projects to improve traffic flow while ignoring the historic design of the bridges and woodsy layout of the terrain. Several miles of highway are protected by aluminum crash barriers that have not weathered well, and do not fit in with the parkway theme.

Recently, some groups have organized against the rebuilding. In June 2005, the Merritt Parkway Conservancy and the National Trust for Historic Preservation, together with the Norwalk Land Trust, the Norwalk Preservation Trust and the Norwalk River Watershed Association, filed a lawsuit against the Federal Highway Administration (FHWA), seeking to downsize the massive new interchange project on the historic Merritt Parkway at Route 7 and Main Avenue in Norwalk, Connecticut. The project was originally designed under the assumption that Route 7 would become a full capacity north-south expressway. But Route 7, the expressway version, was never completely built, so

the plaintiffs argued that the project no longer makes sense.

Future Development

Driving down Interstate 495 in Norton and Mansfield, Massachusetts, you encounter a 20-mile-long straight stretch of six-lane highway with a very wide grassy median. One can only imagine what the same space would look like with woods preserved in the median. An argument against a forested median is that it would be undesirable and unsafe to have deer and other large animals nesting in that space and darting into traffic. But animals have found a way of adapting to suburbia regardless, and one counter-argument maintains that a boring, featureless road can result in more accidents. In terms of land usage, the Interstate 495 right-of-way features a lot of wasted, useless space. If the wide median cannot be filled with trees, a better approach would be a layout similar to the Mass Pike or the first nine miles of Interstate 95 in Rhode Island, east of the Connecticut border, where there is no wide median, and, for safety, the lanes of traffic in each direction are separated by a tall, concrete Jersey barrier.

At some point in the future, all of what is currently unused land will be used. Some estimates say that eastern Massachusetts will be built out by 2050.[2] Wooded highway median strips may start to fulfill a role never intended by designers: that of miniature habitat preserves in areas where the natural habitat has largely been plowed over. Some wooded medians are particularly wide, such as Route 128 in Dedham, and Interstate 95 in Connecticut just west of the Rhode Island border (although, in the case of Route 128, a use was found for part of the land by siting a new prison in the median). These unused tracts feature acres of preserved woods that will likely never be developed. The woodlands are peculiarly sited for preservation, being enveloped by streams of vehicles on all sides. At some future date, when our cities and suburbs are fully built out, we will drive on some stretches of freeways that still appear to be carved out of the forest and the roads will remind us of a time when the land was in a wilder state.

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Photo by Ben Thayer.