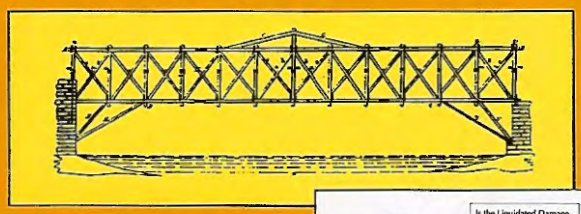
CIVIL ENGINEERING PRACTICE

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

SPRING/SUMMER 1995 VOLUME 10, NUMBER 1 ISSN: 0886-9685



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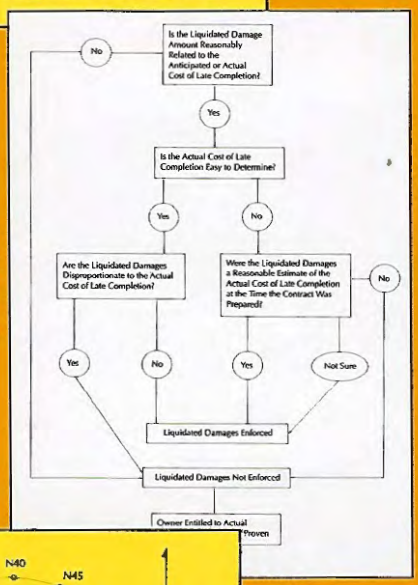

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CIVIL ENGINEERING PRACTICE: JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE (ISSN: 0886-9685) is published twice yearly in the Spring and Fall by the Boston Society of Civil Engineers Section/ASCE (founded in 1848). Editorial, circulation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

Subscription rates are: *U.S.* — Individual, $25.00/year; Library/Corporate, $30.00/year. *Foreign* — Individual, $30.00/year; Library/Corporate, $35.00/year.

Back issue rates for *Civil Engineering Practice* and *The Journal of the BSCE Section/ASCE* are available at $12.50 per copy, plus postage.

Please make all payments in U.S. dollars.

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

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

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Editorial, Circulation & Sales Office:

Civil Engineering Practice
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One Walnut St.
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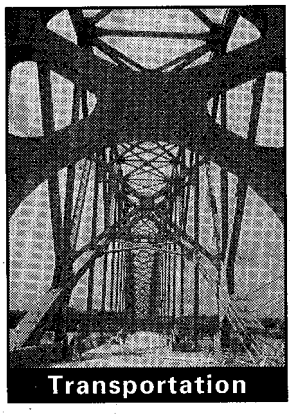
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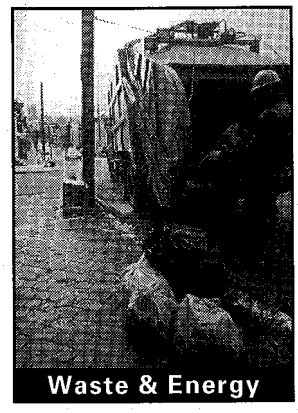
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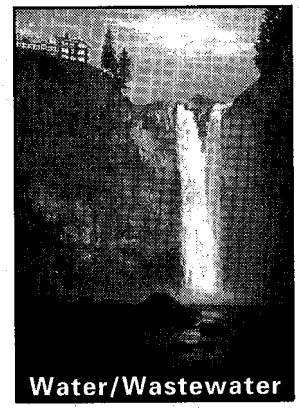
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Editorial

Civil Engineering Practice, as its name indicates, aims to present articles that have a direct bearing on how civil engineers practice their profession. Papers that provide new insights and approaches to common problems, particularly when they have something controversial to say about prevailing practice, are always welcome.

Readers may be surprised to see Richard K. Allen's paper, "The Estimation of Construction Contract Liquidated Damages," which starts on page 7 of this issue. Since its inception, *Civil Engineering Practice* has published only two other papers dealing with the legal aspects of construction contracts ("Basic Contract Law for Civil Engineers" by Sidney Wartel, and "The Architect-Engineer's Role in Design-Build Contracts" by Michael Loulakis, both in the Fall 1987 issue). The journal's editorial board sees Mr. Allen's paper as an inducement to civil engineers involved with construction contracts to present their experiences and express their views in the journal.

As Mr. Allen's paper went through the journal's peer review process, it became apparent that the subject of liquidated damages (and its varied application) is a sensitive one. Hopefully, interested readers will react to it by submitting comments and discussion for publication. Do any of you readers entertain serious doubts about the usefulness of liquidated damages provisions in construction contracts? Do you have a preference for bonus and bonus/penalty provisions? Do you think all such provisions should be negotiable prior to contract signing, rather than decreed by the owner? Can the "partnering" approach obviate the need for penalties and bonuses? After all, isn't it equally advantageous to the contractor and owner to complete the job on or ahead of schedule?

The "Forum" section of *Civil Engineering* magazine in March 1995 presented a severe criticism of a formula developed by the Armed Services Board of Contract Appeals "to compensate a contractor for allegedly lost home-office-overhead income in cases of aggravated government interference in the contractor's operations." The "Readers Write" section of the May 1995 issue of *Civil Engineering* included a response to the March article that provided the reader of both articles with a much fuller understanding of the subject.

By submitting stimulating discussions of articles published in *Civil Engineering Practice,* you can enhance the BSCES journal's value to the profession.

Lee Wolman

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The Estimation of Construction Contract Liquidated Damages

Determining the actual cost of the late completion of a construction project to an owner is key to reducing or avoiding disputes between the owner and contractor.

RICHARD K. ALLEN

There are few businesses for which the saying "time is money" is more appropriate than the construction industry. Construction contracting is an extremely time sensitive activity. An owner can lose opportunity and profits waiting for the completion of a late project. Similarly, when a project extends beyond its scheduled completion date, contractors often carry the financial burden of maintaining field and office personnel longer than anticipated. In today's competitive marketplace, few owners or contractors can afford the cost of a late project.

For many projects, owners shift at least part of the risk of late completion onto contractors. The most common form of risk shifting is the inclusion of a liquidated damages provision in the construction agreement. Liquidated damages are defined in the construction contract and are chargeable against funds due to the contractor for each day the contractor fails to complete the project beyond the contract completion date. Therefore, a liquidated damages provision provides a straightforward way to calculate monies due to an owner if a project takes longer than agreed to complete.

Contract Completion Date

The original contract completion date is the date specified in the contract when the project is required to be finished. Many contracts simply set a number of days that it will take to perform the task (instead of a specific date) in order to accommodate the uncertainty of when a project may be authorized to proceed.

In certain construction contracts, the owner may not only set a final completion date but also require that portions of the work be finished by interim dates. These interim dates are commonly referred to as *milestones*. A contractor may be required to meet milestones for specified portions of the work. A liquidated damages provision may apply to milestone dates as well as the contract completion date. Under these circumstances, it is possible that the liquidated damages could then accumulate through the succession of more than one missed milestone dates through to completion.

EQUATION 1

Completion Date	=	Original Completion Date	+	Change Order Time Extensions	+	Additional Days of Extension for Constructive Changes

Milestone or completion dates can be changed during the course of any contract performance. These changes, when agreed upon between the owner and contractor, usually result in a *change order* to the contract. In addition, contracts can be changed by other means. A *constructive change* is one in which the owner's acts or omissions sufficiently alter the conditions under which the contractor agreed to perform the work so that the contractor is entitled to an equitable adjustment in contract price or time of performance. It is not surprising that owners and contractors differ as to whether a contract has been constructively changed and, if so, what, if anything, is a fair and equitable adjustment for the change.

In sum, the contract milestone and completion dates are the original milestone and completion dates set forth in the contract, as adjusted by change orders and constructive changes (see Equation 1).

Completion of Performance

Having established the contract milestone and completion dates, the owner and contractor must then determine what constitutes *completion*. For most commercial transactions, completion is easily measured and achieved at some definable moment. Take the example of buying a car. Payment is due when the car is delivered to the purchaser. Under this scenario, performance is viewed as "complete" upon delivery of the car. In construction practice, the point at which a project becomes "complete" is rarely so definable and, again, is subject to disagreement between owner and contractor.

Under the traditional principles of contract law, the obligation to pay for performance arises only when that performance is perfectly complete in every aspect. However, perfection is not a feasible measurement of performance in construction practice (or for that matter in engineering or architectural practice) and to relieve the owner from the obligation to pay for any performance less than perfect is neither fair nor equitable. Consequently, construction law has adopted the concept of *substantial completion*.

Substantial completion is achieved when the owner has beneficial use of the project. In other words, substantial completion is achieved at that point in time when the contractor has completed enough work that the owner can take advantage of the project for its intended use. For example, a building may be substantially complete when it is occupied, a treatment plant when it is started up, or a road or bridge when it is open for traffic. In each of these examples the work may not be 100 percent complete but the project's purpose has been achieved.

Again, owners and contractors frequently have different opinions as to when a project has reached substantial completion. Owners tend to put off declaring that substantial completion has been reached as long as possible to attain more complete performance and to delay the release of retainage and the commencement of the project warranty period that commonly occur upon substantial completion. *Retainage* is a percentage of the value of the work already completed, usually on the order of five to 10 percent, that is withheld by the owner as an incentive for the contractor to reach substantial completion and to cover costs to the owner in the event the contractor fails to complete the project. Most contracts require the release of retainage upon substantial completion.

Contractors seek substantial completion as soon as possible for a different set of financial reasons. Obviously, contractors seek access to their retainage. Additionally, contractors seek to free up bonding capacity for new projects — only when older projects reach substantial completion do the contractors' associated bond obligations expire.

Substantial completion does *not* relieve the contractor of completing the project in its entirety, nor does it obligate the owner to pay the entire contract price. Substantial completion,

however, does entitle the contractor to be paid the contract balance minus the value of the work that remains to be performed. This "clean-up" work usually is referred to as a *punchlist*. Therefore, in theory, final completion can be defined as set forth in Equation 2.

EQUATION 2

Final Completion = Substantial Completion + Punchlist Completion

In Massachusetts there are certain statutes that further define substantial completion. For example, Section 39G of *Massachusetts General Law Chapter 30* — which applies to public works projects such as highways, bridges, sewers and water mains — provides the following definition:

> "Substantial completion, for the purposes of this section, shall mean either that the work required by the contract has been completed except for work having a contract price of less than one per cent of the then adjusted total contract price, or substantially all of the work has been completed and opened to public use except for minor incomplete or unsatisfactory work items that do not materially impair the usefulness of the work required by the contract."[1]

Section 39K — which applies to public building projects such as the construction, renovation or repair of any public building — provides the following definition of substantial completion:

> "After the receipt of a periodic estimate requesting final payment and within sixty-five days after (a) the contractor fully completes the work or substantially completes the work so that the value of the work remaining to be done is, in the estimate of the awarding authority, less than one per cent of the original contract price, or (b) the contractor substantially completes the work and the awarding authority takes possession for occupancy, whichever occurs first, the awarding authority shall pay the contractor the entire balance due on the contract less (1) a retention based on its estimate of the fair value of its claims against the contractor and of the cost of completing the incomplete and unsatisfactory items of work. . .
>
> "A certificate of the architect to the effect that the contractor has fully or substantially completed the work shall. . . be conclusive. . ."[1]

Whether for a public works or public building project, the statutory definitions of substantial completion incorporate the concept of the *beneficial use* of the project. Under Section 39G, beneficial use is measured by whether the project has been opened for public use — except for minor items that "do not materially impair the *usefulness*" of the project. Under Section 39K, beneficial use is generally agreed to occur when the owner takes over or possesses the project *"for occupancy."*

Exactly when a project can be beneficially used by the owner is a subjective determination that can be subject to disagreement. Massachusetts statutory law, however, also incorporates an objective standard of measuring substantial completion. For public works projects, substantial completion occurs *no later* than when the project is 99 percent complete based on the original contract price. For public building projects, substantial completion occurs *no later* than when the project is 99 percent complete based on the adjusted contract price. In practice, completion can be determined as part of the process of the contractor submitting and the owner accepting periodic payment requisitions that document a project's progress.

Massachusetts law also recognizes that most projects are monitored by architects or engineers who are called upon and are given the authority by contract to determine project completion as part of the pay requisition review process. Deference is given to an architect's or engineer's decision in this regard. Such a decision is held to be conclusive unless it can be shown that it was arbitrary, or otherwise made in bad faith.[1,2]

Conceptually, when the project reaches substantial completion the owner has enough of

what the owner bargained for to obligate the owner to make payments to the contractor. With regard to milestone dates, if substantial completion falls on or before the contract milestone dates as adjusted by contract changes, then the milestone work can be viewed as having been completed on time. Similarly, if the substantial completion date of the entire project falls on or before the contract completion date as adjusted by contract changes, the project can be considered to have been completed on time.

What happens if the contractor fails to meet a project milestone or contract completion date? Failure to meet a time of performance provision of a contract is a breach of that contract which entitles the owner to its damages. If there is a liquidated damages clause in the contract, then the recourse for this breach of contract is the application of the liquidated damages clause.

Liquidated Damages

Liquidated damages are specified daily charges that are deducted from monies otherwise payable to the contractor for each day the contractor fails to meet a milestone and/or contract completion date. Another way of looking at liquidated damages is that it is the price the contractor must pay per day for working beyond the required completion dates.

Liquidated damages are a contract-based remedy for the late completion of a contract. The terms of these damages must be agreed to by the parties in the construction contract and normally take the following, or similar, form:

> If the contractor fails to complete the work within the contract time or fails to achieve any of the contract milestones, the contractor agrees to pay the owner $X per day as liquidated damages to cover losses, expenses and damages of the owner for each and every day which the contractor fails to achieve completion of the milestone work or the entire project.

The key then to liquidated damages is the value assigned to the per diem cost "X."

The Law of Liquidated Damages

It is no surprise that the imposition of liquidated damages fosters disputes between owners and contractors. These disputes, in turn, can spawn litigation that, in turn, cause courts to write decisions regarding the enforceability of liquidated damages provisions. These decisions then become law that owners and engineers need to bear in mind when drafting liquidated damages clauses.

The basic legal principle of liquidated damages is:

> "Damages for breach by either party may be liquidated in the agreement but only at an amount that is reasonable in the light of the anticipated or actual loss caused by the breach and the difficulties of proof of loss. A term fixing unreasonably large liquidated damages is unenforceable on grounds of public policy as a penalty.
>
> "Comment a: . . . [T]he parties to a contract are not free to provide a penalty for its breach. The central objective behind the system of contract remedies is compensatory, not punitive. Punishment of a promissor for having broken his promise has no justification on either economic or other grounds and a term providing such penalty is unenforceable on grounds of public policy."[3]

Despite what many owners and engineers may believe, a liquidated damages provision may *not* punish a contractor for late completion. The purpose of the provision must be compensatory only. The per diem liquidated damages may not be set at whatever level the owner or engineer believes is necessary to coerce the contractor to complete a project on time. Instead, the owner and engineer must set the liquidated damages at an estimate of what it costs the owner if the contractor is late in completion. Contract remedies, like liquidated damages, may not punish and Massachusetts courts will not enforce a liquidated damages provision that acts as a penalty.[4-8]

The fact that liquidated damages are considered to be compensatory suits their application to substantial completion rather than final completion. Since liquidated damages are an estimate of the cost to the owner of a project that cannot be used as the owner intended when the owner intended (provided the owner normally would be able to beneficially use the project at

substantial completion), substantial completion ordinarily cuts off the owners' rights to continued assessment of liquidated damages against the contractor. However, there are exceptions to every rule. For example, when a contract *expressly* tied liquidated damages until no further work remained to be performed, the Utah Supreme Court upheld the assessment of liquidated damages beyond the substantial completion phase.[9]

How does an owner or engineer know whether the contract-defined per diem liquidated damages are a penalty? Massachusetts case decisions establish a number of guidelines to address this question. To be enforceable a liquidated damages provision must satisfy:

> "[T]he well established principle that the amount of liquidated damages specified in a contract must reasonably relate to the anticipated or actual loss caused by the breach."[6]

Therefore, the liquidated damage amount cannot be arbitrary and must relate to the consequences of late completion. Furthermore:

> "[W]here the actual damages are easily ascertainable and the stipulated sum is unreasonably and grossly disproportionate to the real damages from a breach, or is unconscionably excessive, the court will award the aggrieved party no more than [the] actual damages."[4]

If what the owner *actually* incurs for late completion is much less than what is charged against the contractor as liquidated damages, the liquidated damages provision will not be upheld. Massachusetts law thus allows an after-the-fact comparison between what it actually costs the owner for late completion against what the owner has charged the contractor under the liquidated damages provision.

What if the damages are not easily "ascertainable"? The additional cost of engineering as well as the impact on other contractors can usually be determined when a contractor is late. Engineers invoice the owner for extended professional services and other on-site contractors make claims for delay costs. These costs are quantifiable. Inconvenience to the public, lost opportunity, or an owner's ongoing administrative costs (all caused by late completion), however, may be difficult, if not impossible, to quantify with a reasonable degree of accuracy. Nevertheless, they are real costs to the owner. Under these circumstances, Massachusetts law takes into consideration how the liquidated damages provision was determined *at the time* the contract was prepared. This prospective analysis has been articulated as:

> "[W]here actual damages are difficult to ascertain and where the sum agreed upon by the parties at the time of execution of the contract represents a reasonable estimate of the actual damages, such a contract will be enforced."[4]

What if there is uncertainty whether the original estimate at the time of execution of the contract was reasonable or not? Massachusetts law then defers to enforcing the liquidated damages clause as a contract term to which the parties agreed. For example:

> "We recognize that a liquidated damage provision is appropriate where the harm is incapable or very difficult of accurate estimation. . ."[8]
>
> "When losses are difficult to quantify, considerable deference is due the parties reasonable agreement as to liquidated damages. . ."[7]

If a court can figure out how much an owner has been damaged and if those damages are far less than the liquidated damages charged to the contractor (so that they seem unfair in comparison), then the liquidated damages provision will not be enforced. If a court, however, cannot determine what an owner's actual damages are due to late completion, then the liquidated damages will be upheld as long as the estimate of damages at the time of contract preparation was not unreasonable. Figure 1 illustrates this liquidated damages analysis.

How does all this translate into everyday practice? First and foremost, engineers and owners *must* make a good faith estimate of cost to the owner of late completion at the time the construction contract is prepared.

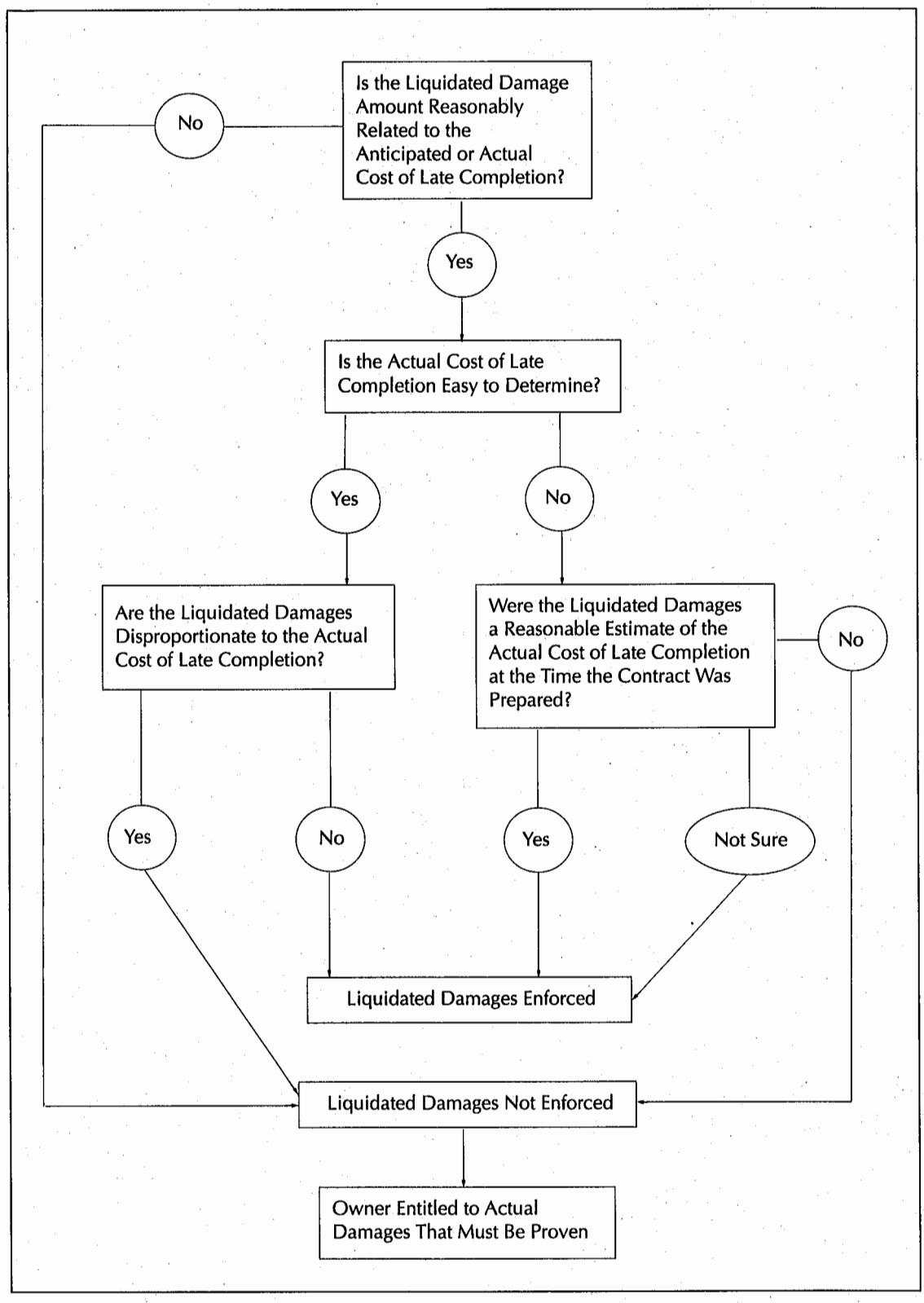

FIGURE 1. Liquidated damages matrix.

Second, engineers and owners must document how that estimate was made. If challenged, the owner must bear the burden of demonstrating that the liquidated damages provision was based on a reasonable forecast of actual damages.[8] Since owners rely on engineers, the engineer would be asked to identify the impacts of late completion and how the cost of each impact was estimated. As a practical matter, documents showing how the amount for liquidated damages was determined are essential. The absence of any documentation may indicate that the specified liquidated damage amount was arbitrarily determined.

EQUATION 3

$$\text{Liquidated Damages} = \frac{\text{Contract Cost} \times \text{Total Extended Cost}}{\text{Total Project Cost} \times \text{Contract Duration}}$$

Project Examples

How the liquidated damage provision was determined for two of Boston's major projects — the Boston Harbor Project of the Massachusetts Water Resources Authority (MWRA) and the Central Artery/Tunnel Project of the Massachusetts Highway Department (MHD) — make for an interesting comparison of approaches and degrees of complexity in estimating liquidated damages.

Boston Harbor Project

The Boston Harbor Project is a billion dollar plus construction effort to construct wastewater treatment facilities that will ultimately clean up Boston Harbor. This project is court-ordered and must meet certain court-established deadlines for completion. Like any large project, the Boston Harbor Project has numerous construction contractors simultaneously working on the same site. In order to coordinate and complete this complex project on time, each contractor must meet milestone and completion dates set in their respective contracts.

The MWRA, through its construction manager, established liquidated damages provisions for the various Boston Harbor Project construction contracts through straightforward proportioning calculations.[10] These proportioning calculations generally involved assigning the engineer's estimate of cost to each construction contract, and then determining the percentage that each contract represented against the total estimated project cost. This percentage would then be applied against what the MWRA estimated would be its "extended cost" during the life of the entire project. These extended costs included the costs of construction management, design services, in-house project management costs, utilities, power, water and the wide variety of support contractors required on-site during the construction period. Each contract was assigned its proportionate share of the MWRA's extended costs based solely on each contract's cost estimate.

In order to determine a per diem liquidated damages amount, the MWRA then divided each contract's proportionate share of the extended costs by an estimate of how long each contract would take to perform. The resulting per diem cost estimate was then used to establish the per diem liquidated damages cost for both contract milestone and completion date breaches.

The MWRA's formulation is provided in Equation 3. Its application is depicted in Table 1.

The MWRA formula inherently assumed that the cost incurred by the MWRA because of late completion is a linear function of contract cost. It is not unreasonable to assume, as a general proposition, that the greater the contract cost, the more resources the MWRA must expend to support that contract. However, construction contracts are not created equal and support costs will not be expended in a linear fashion. For example, support costs on complicated and intensively inspected work would be greater than on more common construction. The MWRA formula does not provide for such adjustment.

It is also not unreasonable to assume that the longer the contract duration, the greater the extended costs will be. Again, however, construction contracts are not created equal. Longer duration contracts may require the ex-

TABLE 1.
Liquidated Damages for the Boston Harbor Project

Contract Package	Contractor	Description of Work	Value ($)	Percent of Total Project Cost	Extended Cost ($)	Duration (Days)	Cost Per Day ($)	Liquidated Damages Per Day ($)
024	Modern Continental	Administration/ Lab Bldg.	53,687,000	2.643	10,070,558	742	13,572	13,000
028	Sciaba	Maintenance Shops	1,915,601	0.094	359,327	152	2,364	2,000
030	Cashman	Till Disposal	14,305,570	0.704	2,683,426	496	5,410	2,000
040	Cashman	Prison Demolition	18,220,000	0.897	3,417,691	757	4,515	4,000
102	JFW/PFK-Mark	N. Main Pump Station	55,838,000	2.749	10,474,040	1,545	6,779	8,000
103	Barletta/Daniel	N. System Headworks	87,123,000	4.290	16,342,452	1,091	14,979	14,000
104	Dick	S. System Pump Station	49,375,000	2.431	9,261,717	1,170	7,916	8,000
105	Gust K. Newburg	Primary Clarifiers	96,997,000	4.776	18,194,607	1,217	14,950	15,000
282	Kiewit	Effluent Outfall	201,919,000	9.942	37,875,779	1,702	22,254	30,000
283	Cashman	Effluent Diffusers	76,770,000	3.780	14,400,445	1,034	13,927	30,000
301	Perini	Residuals for Primaries	160,756,000	7.915	30,154,461	1,293	23,321	22,000
427	Sciaba	Perm. Swgr.	10,551,000	0.520	1,979,147	1,182	1,674	5,000
805	Sciaba	Sludge Transfer	4,062,000	0.200	761,946	390	1,954	1,000
816	Fishbach & Moore	Personnel Facility	1,880,920	0.093	352,821	184	1,918	1,000
901	Sciaba	Construction Utilities	4,112,000	0.202	771,325	354	2,236	2,000
902	Albert J. Welch	Fuel Facility	4,810,000	0.237	902,255	250	3,609	2,500

Note: Contract packages 030, 805 and 816 have liquidated damages amounts significantly less than what the MWRA had estimated. Contract packages 282, 283 and 427 have contract liquidated damages amounts well in excess of what the MWRA had estimated. The remaining contracts cited have liquidated damages amounts between 70 and 118 percent of the estimated amounts and considered within the general range of the estimate.

penditure of more extended costs per day than shorter duration contracts of equal cost simply because of the nature of the work. The MWRA formula does not consider the nature and timing of support cost expenditures during and after the contract duration.

The MWRA will incur certain fixed support costs independent of project progress. The MWRA will also incur certain variable support costs that ebb and flow with job site activity. By uniformly applying these costs across the entire project, the resulting liquidated damages provision does not reflect the significance of when a contract is performed. Certain contracts may be performed during a critical period of project performance and late completion could have a drastic effect on extended costs. Other contracts may not be critical and late completion may be of far less consequence to overall job progress. The uniform application of extended costs does not allow for this type of distinction since each contract is given equal weight with regard to the overall project schedule. In other words, it is more important *when* a contractor is late than how much the contract was estimated to cost and how long it was expected to take to complete.

The MWRA formulation also fails to take into consideration milestone delays. If liquidated damages are applied against each milestone, late completion could conceivably result in multiple liquidated damages being charged. For instance, a project schedule with three milestones, all of which are missed, could cause liquidated damages to be assessed at three times the calculated rate. No matter how reasonable the original estimate of an owner's actual damages are, it is unreasonable to charge three times that estimate and assert that such charges are "liquidated."

In sum, MWRA's proportionate methodology results in expensive contracts (that must be

performed in short periods of time) having disproportionately higher liquidated damages rates than less expensive contracts that could be performed over longer periods of time regardless of the nature of the work or the timing of the work within the entire project schedule. Furthermore, the MWRA's formulation allows a liquidated damages rate to be determined without considering the logic in the construction schedule or the effect of intermediate milestones.

It should be noted that the MWRA's methodology has been challenged.[11] The court found that there were questions of fact that must be tried before the enforceability of the MWRA's liquidated damages formulation could be determined. The case was settled before trial, therefore these questions were never directly addressed.

Central Artery/Third Harbor Tunnel Project

The Central Artery/Third Harbor Tunnel Project (CA/T) is the largest current construction project in New England. Like the Boston Harbor Project, the CA/T Project will have numerous contractors working within interdependent schedules. Similarly, the CA/T Project has an extensive and expensive management and support services burden for the project construction.

Unlike the Boston Harbor Project, however, the MHD determined liquidated damages for the CA/T Project on a case-by-case basis. The MHD examined the scope of each individual contract, determined where that contract fit within the schedule of the entire project and estimated the level of the support services (such as resident engineers, office engineers, field inspectors, and secretaries) that would be needed if the project were to go beyond its contract completion dates. The MHD also considered costs associated with permits, fees, licenses, right-of-way, pest control and, most importantly, evaluated the impact that late completion of milestone or final contract completion dates would have on other contractors.

The MHD also considered the probability that the delay in one contract would have on related contracts. This impact was then classified in three ways:

TABLE 2.
CA/T Project General Conditions Costs

Contract Value $ Million	General Conditions Impact
1-9.9	$500-800 Per Calendar Day Per $5 Million
10-99.9	$800-1,300 Per Calendar Day Per $10 Million
100-210	$600-900 Per Calendar Day Per $10 Million

- No consequence;
- Relating to non-critical activities; or,
- Causing a critical impact to other contractors.

In the event that another contractor was affected, the MHD then estimated the anticipated cost that a delayed contractor could assert based on historical data of "general conditions" costs. These costs are described in Table 2. For example, for an $8 million project the general conditions costs would be on the order of $800 to $1,280 per day.

The MHD then discounted the impact cost based on the probability of impact as measured by available float time between the projects. Finally, the MHD considered project postponement and financing costs by applying cost escalation factors. For example, for CA/T Project package No. CBED9, the MDH calculated liquidated damages for contract milestones at: $2,500 for final acceptance; $9,000 per day for the building ready to receive equipment; and, $500 per day for a complete remediation plan.

There may disagreement with the various rates, probability factors and historical data on which the MHD relied to develop liquidated damages. However, the systematic analysis of impacts on a contract-by-contract basis is rational, considers scheduling logic and requires the exercise of sound engineering judgment.

Conclusion

An engineer must make a reasonable estimate of the owner's damages in the event of the late completion of a milestone or contract completion date in order to apply that estimate as a liquidated damages clause in a contract. The more difficult it is to estimate an owner's dam-

ages, the more likely a court will defer to the liquidated damages provision as agreed to by the owner and contractor. There are a number of ways to estimate liquidated damages.

The MWRA utilized a proportionate application of its extended costs across all contracts regardless of the interdependence between contractor performance, schedule logic and nature of contract work. The MHD estimated liquidated damages on a case-by-case basis using historical data that were adjusted for the probability of affecting other work as well as individual estimates of management and other costs. The MHD's method is more sophisticated and requires the exercise of engineering judgment. The MWRA's method reflects an across-the-board application that is more open to being challenged.

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The Analysis & Design of the Superconducting Super Collider Underground Structures

The design and construction of the complex underground structures for a project of this size and scope posed tremendous engineering challenges.

GORDON T. CLARK & BIRGER SCHMIDT

The Superconducting Super Collider (SSC) project was to be the largest and most powerful particle accelerator in the world. Once it was completed, the SSC would have been the premier high energy physics laboratory anywhere. The original cost of the SSC was approximately $6 billion (later increased to $8 billion). The SSC was under construction 32 kilometers south of Dallas, Texas, until funding for the project was canceled by Congress in 1993 as part of an effort to reduce the budget deficit. This action took place approximately three years into the project. Several of the physical facilities for the project had already been constructed (see Figure 1) and several others were in various stages of design. Of the approximately $1.2 billion allocated for conventional facilities $300 million were spent prior to project termination. It is projected that the cost of environmental work to restore the site and settle claims with contractors and with the state of Texas will significantly reduce, if not nullify, any anticipated federal budget savings.

The SSC was designed to push two counter-rotating beams of protons to nearly the speed of light and then collide them at an energy of 40 trillion electron volts (20 times more energy than possible today). These collisions would have recreated conditions that existed following the "Big Bang" when the universe was less than a trillionth of a second old. At that moment, several different types of subatomic particles called *quarks* existed as individual entities for a few nanoseconds before combining with other particles to form the protons, neutrons and electrons that eventually combined to form the elements identified on the periodic table. Proving the existence of these particles will help scientists better understand how the uni-

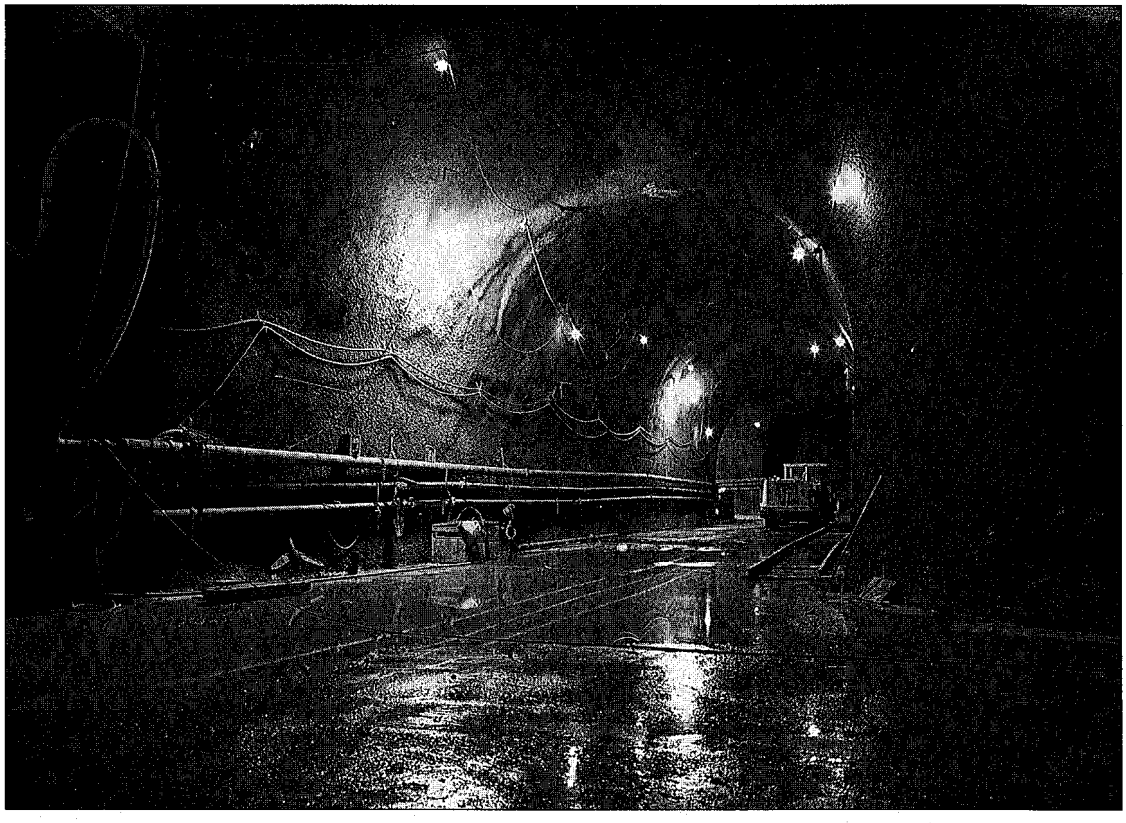

FIGURE 1. Hammerhead adit of the personnel/utility shaft at N25. (Photo courtesy of Benjamin Rodriguez.)

verse was formed, how it evolved, and how gravity and magnetism work.

In simplest terms, the SSC comprised two stainless steel pipes 87 kilometers in circumference. Each pipe was to contain a beam of protons the thickness of a pencil lead, speeding in opposite directions around the main collider tunnel at about 3,400 revolutions per second (near the speed of light). Coils of niobium titanium alloy wire were located outside the 100-millimeter diameter stainless steel pipes. At temperatures just above absolute zero, the wire was to have acted as a superconductor — carrying a high current with near zero resistance. This superconductivity would have allowed the coils to form extremely powerful magnets to guide the proton beams around their oval circuit. The beams would have collided in interaction halls housing special detectors weighing over 30,000 tonnes. These detectors were to use ultra-high-speed computers to sift through the collision debris to detect subatomic particles that exist for less than a billionth of a second. At the time of cancellation, working prototypes of the magnets had been produced and successfully tested. In addition, the design of the detectors was well under way.

The process of acceleration was to begin at a point where the protons were stripped from hydrogen atoms, focused and accelerated in a 256-meter-long linear accelerator (Linac). The protons would then be accelerated further by the low energy booster (LEB), a small horizontal circular accelerator with a diameter of 180 meters. The speed of the protons would be further increased as they traveled through a straight connecting tunnel and around a larger circular accelerator ring, the 1.25-kilometer diameter medium energy booster (MEB).

The Linac, LEB and MEB were to use non-superconducting (warm) magnets to accelerate and guide the proton beams. From the MEB, the protons were to travel through another straight tunnel into the high energy booster

(HEB), which would have accelerated them further as they traveled around its 3.5-kilometer diameter ring. The protons would finally enter the main collider ring, where they would be accelerated to near the speed of light. The proton beams would collide inside the detectors in the interaction halls. The interaction halls were to be located on the East and West Campuses. The different components of the SSC are shown in Figure 2.

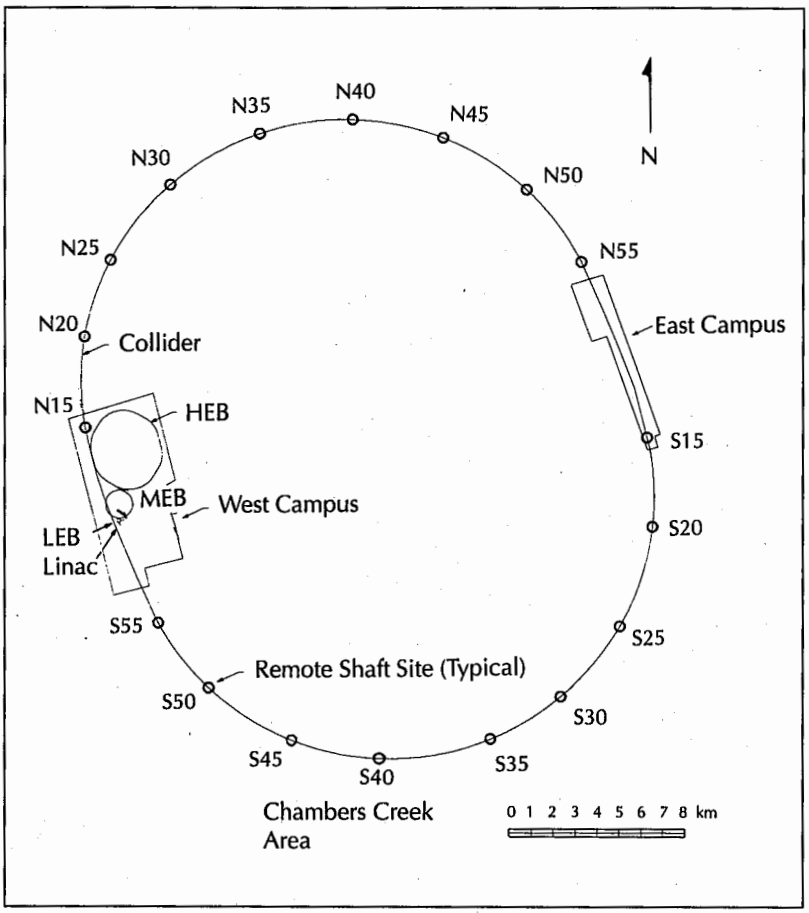

FIGURE 2. Components of the SSC.

The Linac, LEB, MEB, HEB, main collider ring and interaction halls were to be constructed underground in tunnels and caverns in order to provide a stable base for the equipment and to shield the accelerated particles. The SSC project represented over 112 kilometers of tunnels, 60 vertical shafts and four underground interaction halls (two halls at each campus) each the size of a football field. The tunnels, shafts and halls were to be constructed at depths ranging from four to 70 meters below the surface.

To accomplish the anticipated construction would have required the excavation of over 5.5 million cubic meters of rock and the placement of almost one million cubic meters of concrete and shotcrete. In addition, over 150 kilometers of rock bolts and over 100,000 tonnes of structural and reinforcing steel were to be used.

Construction of the underground structures was broken into two distinct contract types:

- Basic contracts, which included excavation and initial ground stabilization; and,
- Finish contracts, which added final linings, placed utilities and constructed surface support buildings.

Site Geology

The site south of Dallas was chosen over several other sites based on search criteria that included:

- The estimated cost of construction/operation;
- Proximity to major universities;
- Available multiple modes of transportation;
- Minimal environmental impacts;
- Stable geology; and,
- Proximity to a major manufacturing center.

At first glance, the geology of the Dallas site offered a massive formation of Austin Chalk (AC) and Taylor Marl (TM) in which to construct the project's tunnels and underground experimental halls. Historically, the area also had a very low probability of earthquakes. The

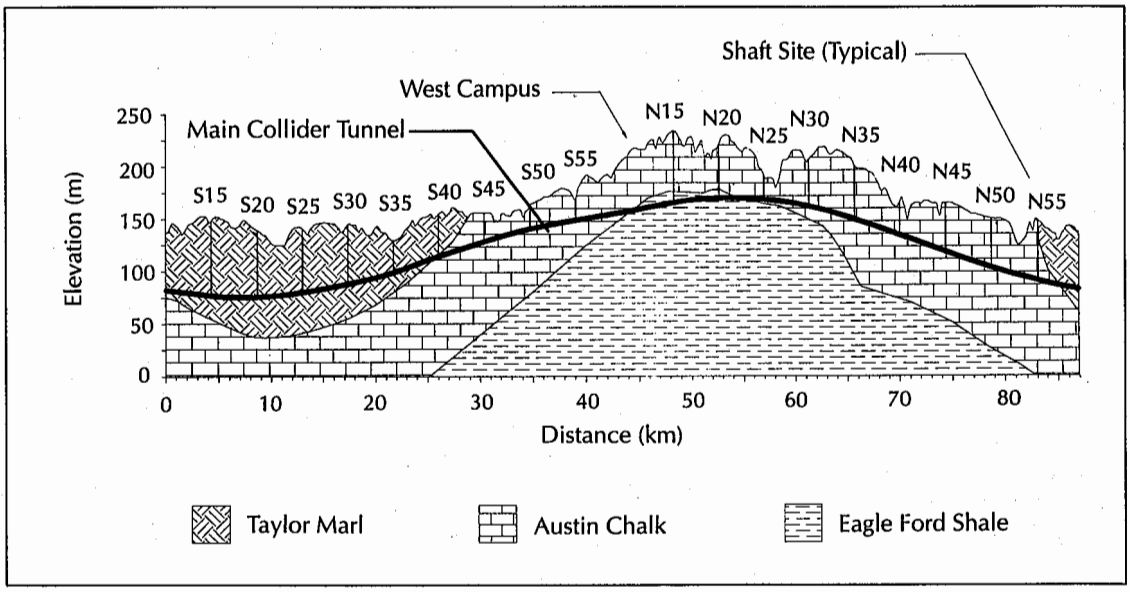

FIGURE 3. Geological cross section of the main collider tunnel and shafts.

marl, while weaker than the chalk, was thought to be an adequate material in which to construct the work.

After the site selection was made, further investigations revealed that not all of the tunnels and halls could be contained in the two formations. A small portion of the tunnel and several shaft bottoms would extend into a formation known as Eagle Ford Shale (EFS). This shale is quite different from the marl or chalk. It has much lower compressive strength and a tendency to expand (swell) and crumble (slake) in the presence of water or humidity. These characteristics posed a difficult challenge to the designers of the underground structures.

From a tunneling point of view, the geologic conditions at the site are relatively straightforward: three rock types prevail along the entire length of tunnel, each of them relatively uniform in character. From the top, the TM, the AC and the EFS — all Cretaceous formations — underlie the site. The layers dip slightly to the southeast. The TM outcrops in the east and is the tunneling medium in the southeast. The AC outcrops over the rest of the site, and most of the tunnel is in the AC. In the west, the underlying EFS forms the tunneling medium. Figure 3 shows the SSC ring tunnel alignment with respect to the geologic strata. Table 1 shows the length and depth of tunnel in each stratum.

Bedrock was generally covered by a thin layer of residual soil that did not enter into design considerations. Note that the plane of the tunnel was dipped slightly to take advan-

TABLE 1
Breakdown of Collider Tunnel in Each Geologic Stratum

Geologic Stratum	Length of Tunnel (km)	% of Total Tunnel	Depth to Crown (m)
AC	50.5	56.6	10-65
TM	27.7	31.6	35-75
EFS	6.6	7.6	40-75
Mixed Face (EFS/AC)	3.0	3.4	20-65
Mixed Face (AC/TM)	0.7	0.8	30-35

tage of the geology and maximize the length of the tunnel in the chalk.

The site landscape is of low relief, with a slope to the southeast toward the Gulf of Mexico. Several river channels cross the site, and Chambers Creek in the southwest (between S40 and S45 on Figure 2) cuts as close as 10 meters to the tunnel crown. In the southeast, Lake Bardwell, a reservoir on the Waxahachie Creek, overlies the tunnel by over 30 meters in the TM. Weathering reaches a depth of up to seven meters in the AC and up to 12 meters or more in the TM. A few faults in the AC are associated with shear zones that carry weathering down to 20 meters or more. The tunnel was to intercept one or more of these weathered shear zones.

The EFS is a weak shale with a compressive strength of about 1.2 MPa at the top, increasing with depth to over 2.4 MPa at the deepest tunnel elevations. Compared to the compressive strength, the tensile strength (as determined by the Brazilian method) is abnormally high — 0.7 to 1.1 MPa. Soil-type laboratory tests were performed on this material, and it was found that it reasonably obeys the laws of soil mechanics (effective stress theory, consolidation and strength characteristics). It was also found that it slakes badly upon exposure. After it has been air dried, it decomposes completely and quickly when immersed in water. On the other hand, it decomposes only partly (some 20 percent) when immersed fresh and not dried.

This fact, together with the finding that some consolidation samples, when exposed to high strains, rebounded to a volume much greater than the initial volume, led to the conclusion that the in-situ material properties are very much affected by cementation. The approximate five percent calcite content suggested that the cementation is by calcite. The cementation appears to be destroyed by excessive deformation or by drying and other exposure.

When exposed at a typical tunnel depth of 70 meters, the EFS would theoretically be overstressed and subject to plastic behavior. The nature of such behavior for this material was very much in doubt, and it was decided to conduct in-situ experiments in an exploratory shaft.

The AC has the characteristics of weak concrete (unconfined compressive strength of 14 to 20 MPa) and does not appear to be affected by long-term displacements or creep. It would generally not be overstressed. The AC is layered, with frequent, thin shaley layers and occasional bentonitic seams over 30 centimeters in thickness. It has few discontinuous joints and is described, in spite of the layering, as a massive rock. Most people would consider it an ideal tunnel medium.

The TM is a calcareous shale with a range of compressive strength from 1.5 to 5 MPa. It is generally massive and would be generally not overstressed during construction, when the "undrained" behavior has to be considered. Again, soil-type tests suggested that the long-term strength would be lower and could result in local overstress and plastic displacement. A utility shaft was instrumented to assess both the short-term modulus of deformation and the long-term effects.

Exploration Program

Initial exploratory borings were made during the time the site selection process was being conducted. These borings were supplemented by additional location-specific borings carried out before the design/construction contract was awarded. During the detailed design phase, the design/construction team conducted further design-specific explorations. A number of exploratory tools were employed during the course of the work.

Seismic profiles were run in areas of specific interest, such as in the Chambers Creek area, where determining the top of the rock was essential. This method was generally unsuccessful in determining fault locations and stratigraphic horizons, largely because the velocity contrast across the boundaries was too small. However, it was useful in determining the top-of-rock depths below the relatively thin surface soil.

Downhole geophysical surveys were conducted in a number of boreholes. These surveys were successful in determining the signatures of strata sequences in the AC. They also helped correlate data between boreholes (which is not possible by core examination alone) and thus indicated potential fault locations. In particular, electric resistivity and natural gamma surveys clearly marked the clay mineral content of the strata.

Numerous near-surface borings were drilled to determine the characteristics of residual soils, weathered rock and alluvium for surface structure foundations and infrastructure (roads, utilities, etc.). These borings had no impact on the design of the underground structures.

The greatest effort was expended on core borings. NX-size as well as three-inch and Pitcher barrel cores of several diameters were recovered. At the outset, it was thought that Pitcher barrel sampling would be the best method to recover undisturbed EFS and TM samples for testing, largely because the friction between samples and core barrel is greatly reduced. The Pitcher barrel is designed to cut a sample slightly smaller in diameter than the inside of the core barrel.

As it turned out, initial testing indicated abnormally low strength and modulus of the EFS when Pitcher barrel samples were tested. It was concluded that the small space between barrel and core permitted softening of the core by water trapped in the space, resulting in low strength data. These tests were discarded and Pitcher barrel sampling discontinued.

Pressuremeter tests were conducted in two borings with doubtful results. On the other hand, hydrofracturing tests were apparently successful in determining the in-situ stress regime. It was concluded that the horizontal stress in the AC is up to about twice the overburden pressure, while in the EFS the horizontal stress is up to about 1.5 times the overburden. By conjecture, it was assumed that the TM horizontal stress fell within the same range.

In-situ packer tests using an instrumented packer (electronic pressure measurement in the packer interval and below) were conducted in many boreholes to assess the permeability of the rock. For the most part, the permeability was near the lower limit of the accuracy of measurement (some 10^{-8} cm/sec).

At the outset, boring spacings of about 600 meters, with secondary borings cutting the spacing to 300 meters or less depending on the results of the primary borings, were planned for most of the underground structures. One boring was placed at each shaft. As time went on, however, the budget tightened and at the same time the designers' confidence increased. Therefore, the last stretches were explored with boring spacings greater than one kilometer.

One exception was the Chambers Creek area, where borings were about 50 meters apart, drilled to ascertain the top of rock and depth of weathered rock in this area of very shallow tunnel. Another exception was the western region, where injector tunnels from the HEB were to join the main collider tunnel from above and at a low angle. These complex intersections are found in the EFS with some parts in the AC.

In increasing the spacing of borings, it was recognized that the knowledge of the geologic conditions in situ became more uncertain, which was likely to result in greater contingency in contractors' bids and greater risk of claims for differing site conditions.

Special Tests in the Exploratory Shaft

While the AC forms a benign environment for underground works, the EFS is potentially treacherous. With a strength near the AC/EFS interface as low as 1.4 MPa and an overburden pressure in the deeper portions of the collider tunnel exceeding 1.4 MPa, stress concentrations around underground openings (shafts, tunnels) would cause significant overstress.

The behavior of a soft shale under these overstress conditions is not readily predictable based on theory and laboratory tests. In the laboratory, this material appeared to behave more like a soil than a rock, obeying laws of consolidation and swell, and effective stress and strength theory. It was judged that the only way to assess actual, in-situ behavior of the EFS was to perform full-scale in-situ tests in an exploratory shaft.

The exploratory shaft was completed by early 1992 at a location on the west side where the depth to the AC/EFS interface was about 65 meters. The five-meter-diameter shaft was drilled to a depth of 60 meters. Drilling was stopped at that point to permit the installation of instruments in the EFS below, including inclinometer casings to permit measurement of lateral displacement as well as piezometers to measure pore water pressures in the EFS. Then, drilling was continued at three meters in di-

ameter for another 20 meters (about 16 meters into the EFS).

Shaft drilling was performed using a converted off-shore oil derrick with a substructure that permitted inserting large drill tools. These drill tools included a 75-centimeter auger to drill a pilot hole, followed by a 2.5-meter bucket excavator following the pilot, then a bucket with reamer arms to enlarge the hole to at first five meters and later to three meters in diameter. Ground support in the top 10 meters of the AC consisted of shotcrete and wire mesh anchored with 85-centimeter dowels. After examining the excellent quality of the AC, it was decided to eliminate the shotcrete and a two-part latex epoxy sealant was used in the lower part of the AC.

To permit observing how the EFS was performing, it was planned to apply only a coating of the epoxy sealant to the EFS (safety would be assured by a steel cage inside the shaft). The EFS began to spall and slake badly only a few hours after exposure, requiring the application of shotcrete for support. It appeared that stress concentrations around the shaft in the EFS, theoretically up to 2.8 MPa, resulted in the gradual formation of shear fractures beneath and parallel to the wall.

Water seeping in through fractures in the AC and then down the shaft wall undoubtedly contributed to the deterioration of the EFS shaft wall. The rate and timing of seepage appeared to correspond to rainfall in the area and the source of water was thought to be surface runoff rather than groundwater.

Confirmation of the stress effects came when horizontal instrumentation boreholes were drilled into the side walls. The double stress concentrations at the top and bottom of these boreholes resulted in rapid spalling, and the shape of the boreholes rapidly resembled a vertical oval, arrested at the top and bottom by horizontal partings in the shale.

Shaft wall movements in the EFS, as observed in the inclinometer casings, followed quickly upon excavation. The maximum movement occurred near the top of the EFS, where the shale is the softest and where instability problems caused considerable deterioration before being checked. The long-term lateral displacement rates (after the application of 10 to 15 centimeters of shotcrete) are negligible. On the basis of these data, the elastic modulus of the EFS in the horizontal direction was judged to be at least 1,000 MPa, or about three times the laboratory value.

Engineering Properties of the Site Geology

As stated earlier, the AC is comparable to a low-strength concrete while the TM and EFS are much weaker materials. If an underground structure at a depth of 65 meters is considered (with a typical stress concentration factor of two to three), it can be seen that the stresses around an underground opening would be approximately 3.5 MPa. This stress level would indicate overstressing at structures in the EFS, but not in the TM or AC. Stress in the TM could exceed compressive strengths at connections between tunnels and adits where stress multipliers are greater. Thus, the ground around openings in the EFS or TM could experience the formation of plastic or fractured zones as the rock attempted, over time, to accommodate the induced stresses that exceeded its strength.

In contrast, for structures in the AC, the stresses were comfortably below its compressive strength, and the behavior around underground openings in the AC is expected to be well within the elastic range. Table 2 shows the engineering properties used in the analysis and design of underground structures for the SSC.

Types of Underground Structures

The Linac and LEB tunnels were to be constructed using the cut-and-cover technique to depths of 10 to 15 meters and consist of cast-in-place reinforced concrete boxes measuring four by three meters with a wall thickness of 300 millimeters. These excavations would occur exclusively in the AC and would require no bracing during excavation.

The MEB had two shapes of tunnels: a three- by four-meter horseshoe shape that was to be mined with a roadheader and a 4.25-meter diameter circular shape to be bored with a tunnel boring machine (TBM). The tunnel depth ranged from 15 to 20 meters.

The HEB tunnel was designed to be circular and be bored using a TBM with a diameter of

TABLE 2
Geotechnical Design Parameters

Design Parameter	Units	TM	AC	EFS
Saturated Weight (δ_{sat})	kg/m^3	2,000	2,200	2,250
Compressive Strength (U)	MPa	0.06Z+1.2*	15	2
Cohesion (c)	MPa	0.7-1.7	4.4	1.4
Friction Angle (ϕ)	Degrees	10-20	30	0
Deformation Modulus (E)	MPa	830	3,500	830
Permeability	cm/sec	5×10^{-7}	5×10^{-7}	1×10^{-7}
In-Situ Stress Ratio (K_o)		1.4	2.0	1.5
Poisson's Ratio (υ)		0.3		0.2

* *Z* is the depth below fresh TM (usually 10 to 20 meters below surface)

five meters. Depth ranged from 20 to 55 meters. The HEB tunnel was to be cut exclusively in the AC and unlined.

The main collider tunnel was being mined using various TBMs with diameters ranging from five to 5.5 meters, depending on whether

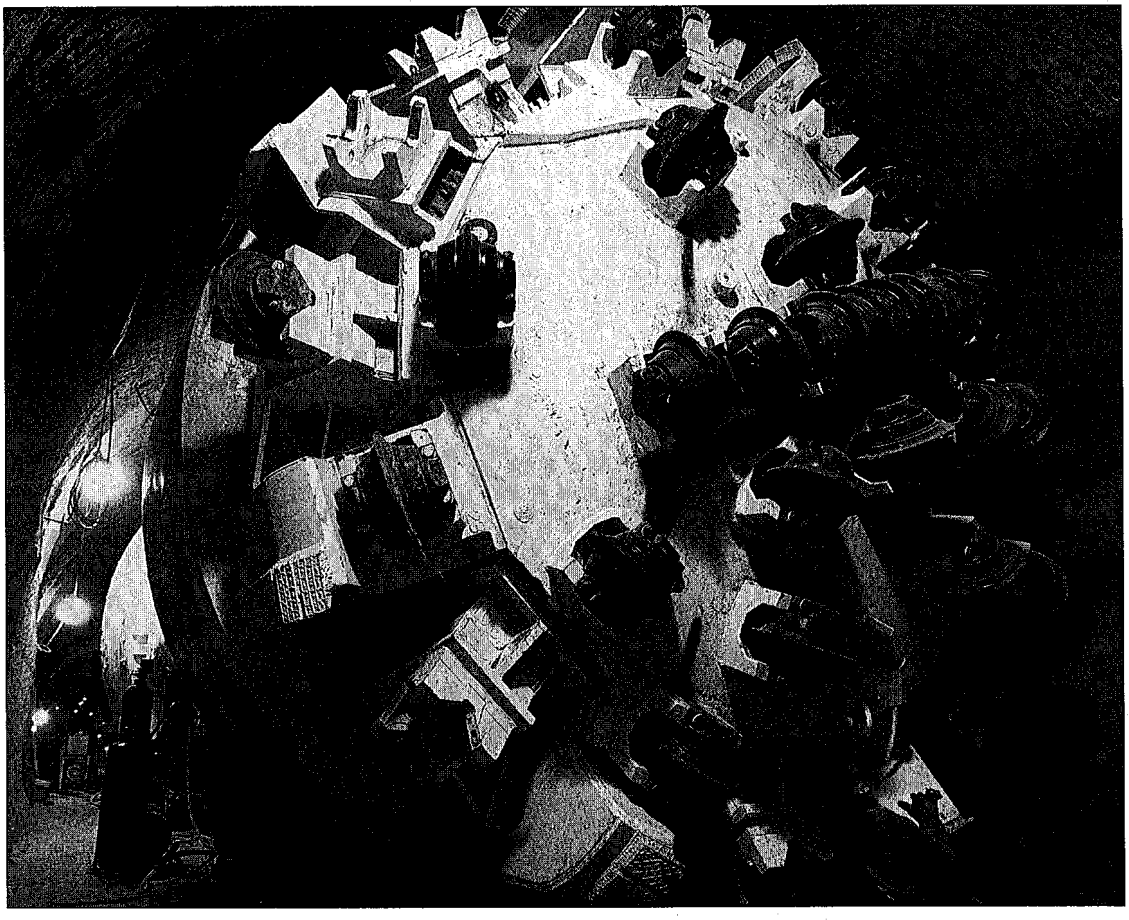

FIGURE 4. TBM used to mine the tunnel from N25 to N35. (Photo courtesy of Benjamin Rodriguez.)

a lining was required. Figure 4 shows the TBM used to bore the N25 to N35 tunnel. Depths of tunnel ranged from 25 to 70 meters. Approximately 25 kilometers of the tunnel were bored before the project was canceled.

Tunnels in the EFS and TM were lined with precast concrete segments placed in the tail of a shielded TBM and then grouted. Tunnels bored in the AC were unlined but had two rock bolts in the crown, as shown in Figure 5. Additional bolts in the crown and a thin layer of shotcrete were used to control unstable rock existing in isolated locations along the alignment.

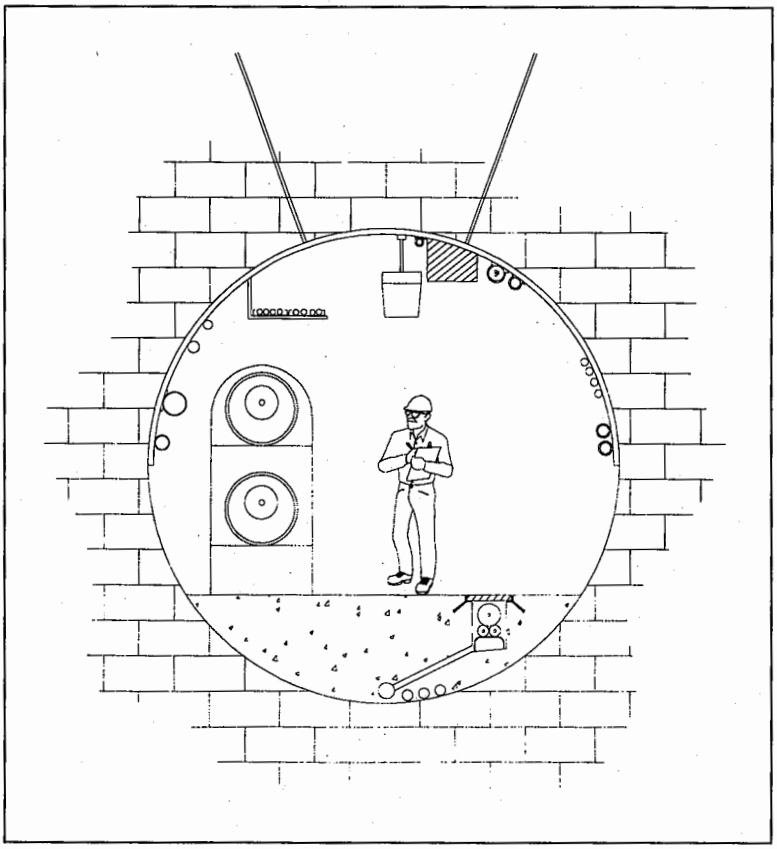

FIGURE 5. Finished collider tunnel in the AC.

The construction schedule for the shafts and tunnels was very tight and required several TBMs to operate simultaneously. This approach required a satellite-based global geodetic control system to ensure that not only tunnel segments lined up with each other, but that all segments across a 1,300-square-kilometer area lay in the same spatial plane when tunneling had been completed. The contractors were given a 250-millimeter diameter bullseye for alignment tolerance. Boring rates ranged from 30 to 145 meters per day. Alignment of the more than 25 kilometers of tunnel bored before project cancellation was generally within a 100-millimeter diameter bullseye. The alignment exceeded the bullseye in a few isolated locations which required minor excavation to correct the fault.

Four main types of shafts were to be used to connect the LEB, MEB, HEB and main collider tunnels with the surface. These shafts were to be constructed to depths ranging from 10 to 70 meters. Each type of shaft serves a specific purpose. Personnel shafts were to house elevators and emergency stairs. Ventilation shafts were to supply fresh air to the tunnel and, also, to house staircases. The utility shafts were to contain cryogenic pipes, high voltage cables and other utilities.

The personnel and utility shafts generally shared a common adit (parallel to the tunnel), which was joined in the middle with another adit connected to the main collider tunnel. All three shafts were circular in cross section with diameters ranging from five to seven meters. In addition, these shafts were offset to the inside of the tunnel by 20 to 50 meters, depending on surface site constraints. They were connected to the tunnel by 10-meter diameter adits. A typical ventilation shaft is shown in Figure 6.

The shaft adits and alcoves were to house cryogenic equipment used to support the cold magnets. A few shafts had diameters of eight meters to permit movement of the TBMs from the surface. In general, the TBMs entered through magnet delivery shafts.

In addition to the three types of circular shafts, there were five in-line elliptical magnet delivery shafts that were going to be used to

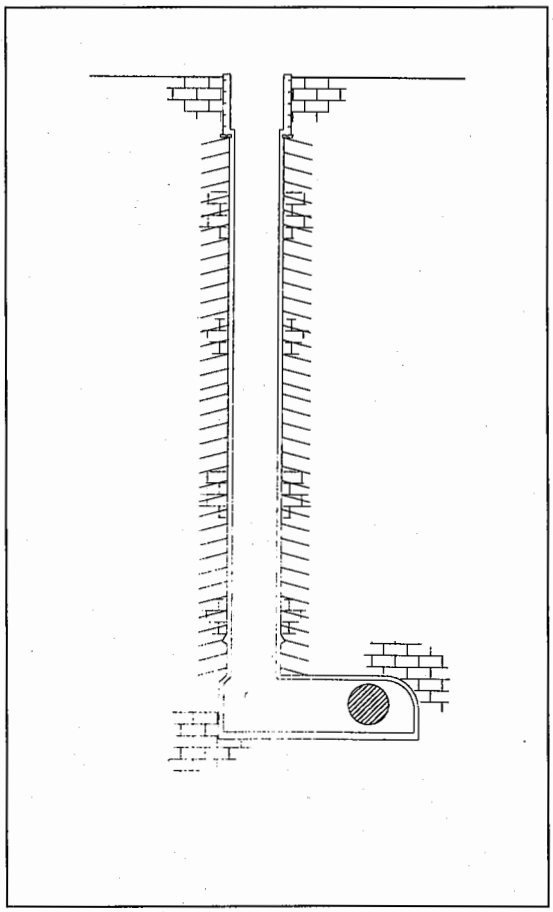

FIGURE 6. Collider ventilation shaft.

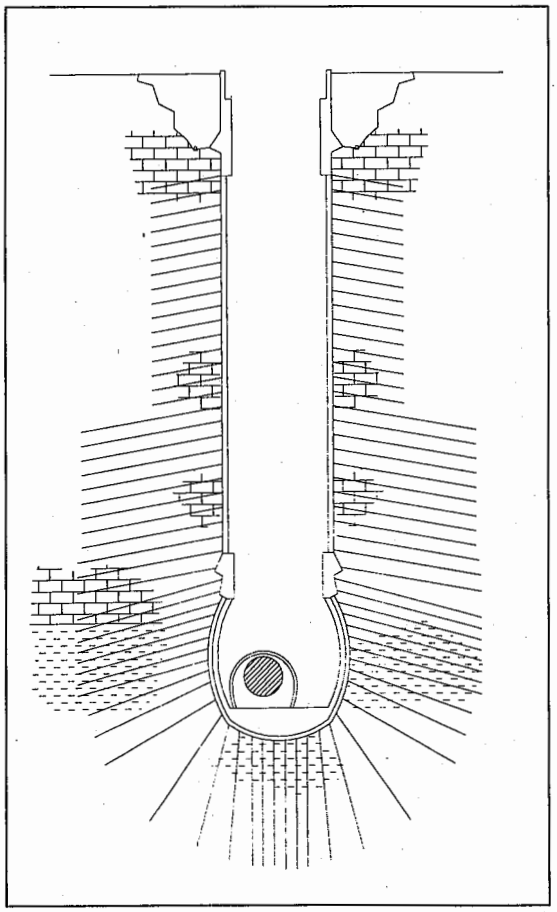

FIGURE 7. Magnet delivery shaft.

lower the cryogenic magnets to the main collider tunnel. These shafts had excavations with a major radius of 20 meters and a minor radius of five to 11 meters. The basic excavation of the magnet shafts at the N15, N40, N55 and S40 sites (see Figures 2 and 3) was complete at the time the project was canceled.

A cross section of the magnet delivery shaft at N15 is shown in Figure 7. This shaft was over 65 meters deep and bedded in the EFS. This shaft was the first constructed (after the exploratory shaft) and required extensive rock bolting to stabilize the shale. A massive finish liner of reinforced concrete was to have been placed to resist the anticipated long-term swell pressure of the shale.

The underground interaction halls that were to house the particle detectors were to have been excavated to a depth of 67 meters using a cut-and-cover method. These halls' floor dimensions were to be 33 by 110 meters. Drilled caissons and permanent cable tiebacks up to 50 meters long were designed to support the vertical sides of the excavation. A reinforced cast-in-place concrete floor slab three meters thick and steel roof trusses 15 meters deep were to be added before backfilling 10 meters of rock over the roof. A preliminary design of one of the interaction halls is shown in Figure 8. Several shafts were to provide access to the interaction halls for personnel, equipment and utilities.

Analysis Techniques

Analyses of the stresses in the rock and structural liners of the shafts at the various stages of construction and life cyle of the structure were needed to develop economical designs and assure the stability of the excavations. These analyses needed to consider both in-situ rock conditions at the various strata and the antici-

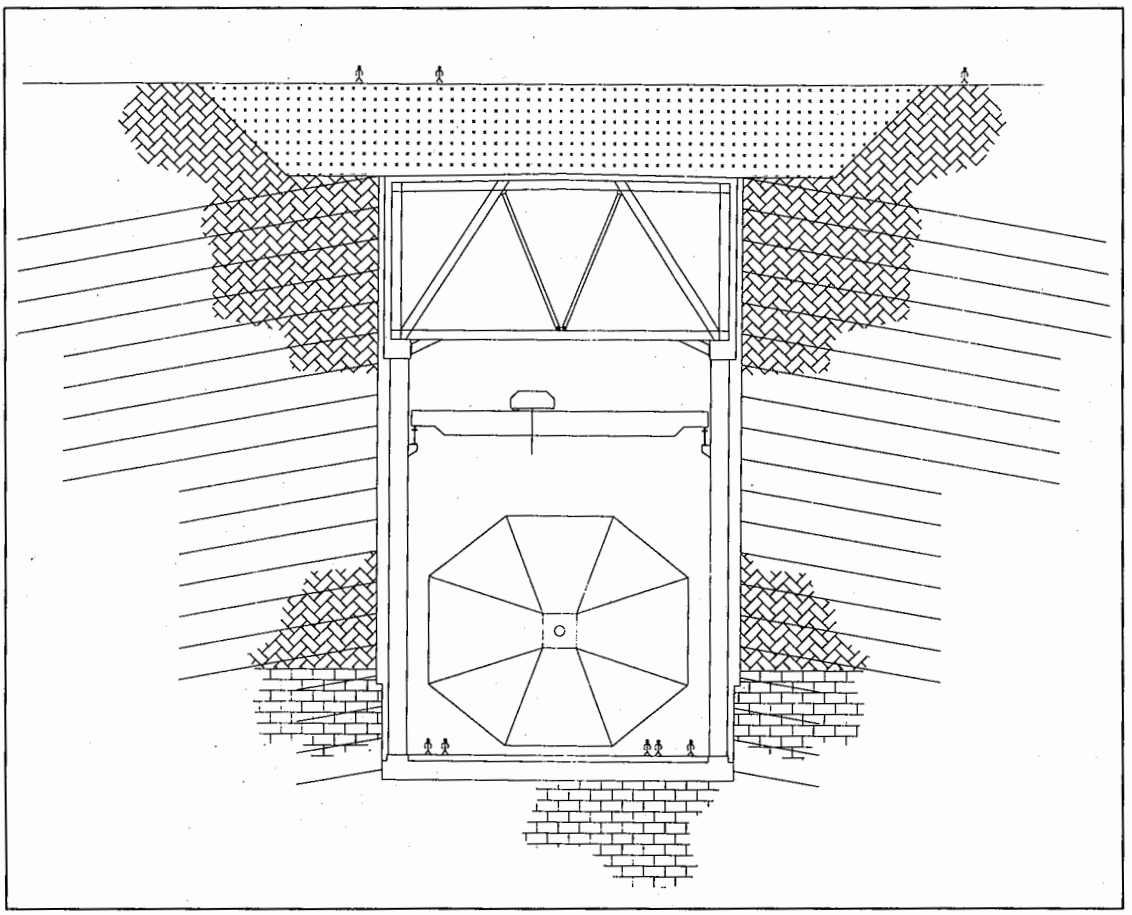

FIGURE 8. Preliminary design of the interaction hall for the gluon-electron-muon detector.

pated construction sequence. Since the underground structures were being constructed in three geologic strata, unique consideration and design assumptions were required.

While traditional closed form solutions were used to check the reasonableness of the design approach, it was felt that state-of-the-art finite element computer models were needed to analyze the rock-structure interaction since plastic behavior was expected. This interaction was complicated by both the geometry of the excavations and the three distinctly different geologic strata. The closed form solutions were primarily used to check the results of finite difference and finite element models.

Two different computer analysis software programs were used:

- A personal-computer-based two-dimensional finite difference program; and,
- A minicomputer-based three-dimensional finite element program.

Both programs had non-linear analysis capabilities. The results of the two programs were compared and closely agreed on predicted stresses and deformations of the circular shafts and tunnels.

The three-dimensional program was used to analyze the more complicated geometry and excavation of the shaft bottoms and adits. The three-dimensional finite element mesh for the analysis of the shaft and adit liners at the N15 utility shaft is shown in Figure 9. The basic excavation of this shaft was completed before project cancellation. This shaft was bedded in the EFS and excavation caused stresses that exceeded its elastic strength. The program took into account the effect of the plastic zones on stress redistribu-

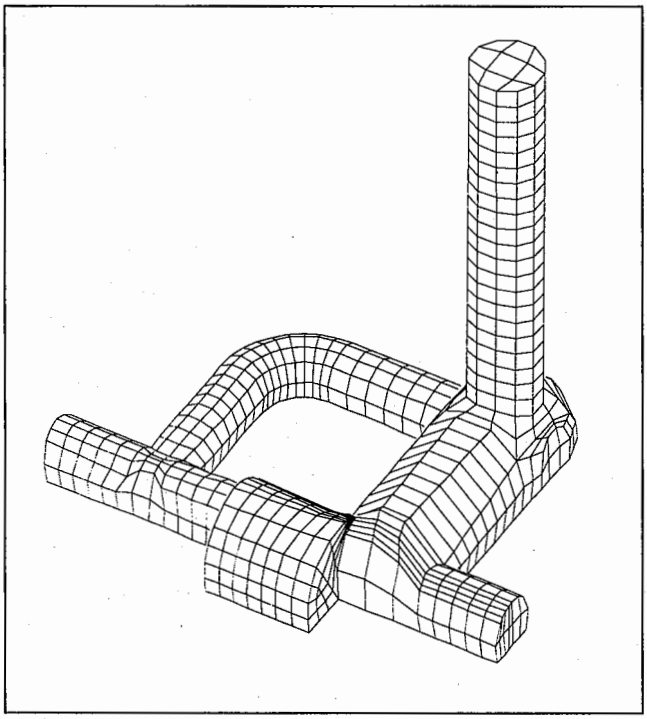

FIGURE 9. Finite element model of the utility shaft liner at N15.

tion and the excavated shaft performed as expected with the analysis results being used to locate additional rockbolts and shotcrete in high-stress areas.

Tunnel Analysis. The Linac, LEB and portions of the MEB tunnels were constructed using the cut-and-cover excavation technique. Excavation loads due to surface alluvial soils were considered. However, due to the strength of the AC, the structures did not require a structural support system to resist horizontal earth loads. The structural box sections were analyzed, using a traditional approach, for vertical earth pressure and hydrostatic loads.

The HEB and portions of the main collider tunnel to be bored in the AC did not require a structural support system due to the strength of the rock and the fact that no long-term deformation was anticipated. It was expected that some loosening of the AC in the tunnel walls and crown (particularly at shaley seams) might take place. Therefore, a local support system consisting of cemented dowels was used. In locations where even greater local instability might be encountered, it was thought to use shotcrete and a more extensive use of dowels.

A precast invert segment was designed and used to transfer construction loads due to the movement of equipment and materials prior to placement of the final cast-in-place invert slab.

As previously stated, the AC is expected to behave in an elastic manner, allowing the use of closed form solutions or simplified computer models. The closed form equation for radial displacement of an unlined tunnel in an elastic-plastic frictionless medium in which a plastic zone may develop is given by:[1]

$$\delta_r/r = c[(1 + \upsilon)/E]\, e^{[(P/c) - 1]}$$

where:

δ_r = Change in radius
r = Excavated radius
c = Cohesion
υ = Poisson's Ratio
E = Modulus of deformation
P = Pressure in soil

The closed form equations for radial displacement of an unlined tunnel in an elastic-plastic medium with friction and in which a plastic zone develops is given by:[1]

$$\delta_r/r = [(1 + \upsilon)/E]\sin\varphi(Pz + c\cot\varphi)[(1 - \sin\varphi)(Pz + c\cot\varphi)/(Pi + c\cot\varphi)]\, e^{[(1 - \sin\varphi)/\sin\varphi]}$$

where:

Pz = Pressure in soil
Pi = Pressure inside tunnel
φ = Angle of friction of medium

The portions of the collider tunnel bored in the EFS and TM, as well as the mixed face with the AC, required more sophisticated analysis techniques that accounted for the formation of plastic zones in the ground surrounding the tunnel excavation. After excavation, the EFS would not be strong enough to support ground pressures and a support system would be required to ensure stability and to control long-term deformation. The support system was to be carried through the entire tunnel length to

allow the TBM to shove off the lining and advance. In addition, it was anticipated that the EFS and TM would exhibit creep behavior over time.

This phenomenon was also considered in the analysis. The tunnel liner segments were analyzed for loads due to material handling, erection, TBM jacking, and initial and long-term soil pressures. The closed form solutions did not account for the in-situ stress ratio, K_0, being greater than 1.0, but were used to check the order of magnitude of the results of the general computer models.

The two-dimensional analysis software was used to develop a model of the tunnel and surrounding medium. The model represented a tunnel transverse cross section. Since most finite element codes are sensitive to boundary conditions, the model represented the EFS surrounding the tunnel to a distance of five tunnel diameters. Boundaries were fixed, an in-situ soil pressure was applied that represented a gravity load parallel to the vertical axis, and a gravity load multiplied by K_0 was applied parallel to the horizontal axis. A typical mesh used to analyze the tunnel excavation is shown in Figure 10.

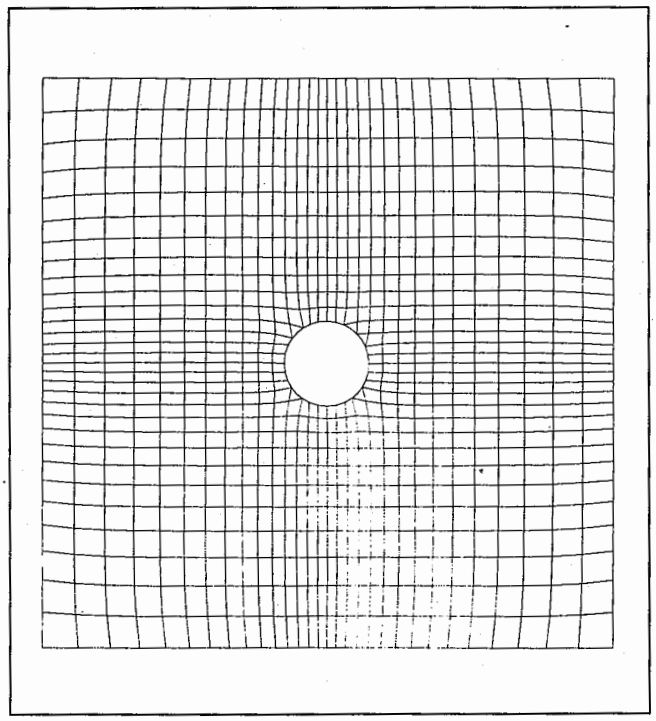

FIGURE 10. Typical two-dimensional analysis mesh of the tunnel.

The three-dimensional software was used to check the two-dimensional software results. The three-dimensional software is based on a sophisticated algorithm that permits generating complicated models that incorporate coupled pore water pressures and the adjustment of the dilatency angle. This code was used for the analysis of shafts and adits, where complicated geometry required use of three-dimensional analysis.

To test the validity of the tunnel models, an initial value for K_0 equal to 1.0 was used. The predicted deformation values were compared to the closed form solutions for an unlined tunnel at a depth of 70 meters. Table 3 shows that these values compared closely. The difference in results of the solutions for the drained case (as shown in Table 3) is due to limitations in the closed form solution that do not account for volume change in the plastic zone surrounding the excavation. These limitations resulted in a smaller predicted deformation.

Shaft Analysis. The analyses of the shafts in the different geologic strata, the varying depths

TABLE 3
Comparison of Tunnel Analysis Results

Radial Deformation of Unlined Tunnel (5 m Diameter)			
State	**Closed Form**	**Two-Dimensional**	**Three-Dimensional**
Undrained	2.3 cm	2.4 cm	2.4 cm
Drained	6.6 cm	7.1 cm	7.2 cm

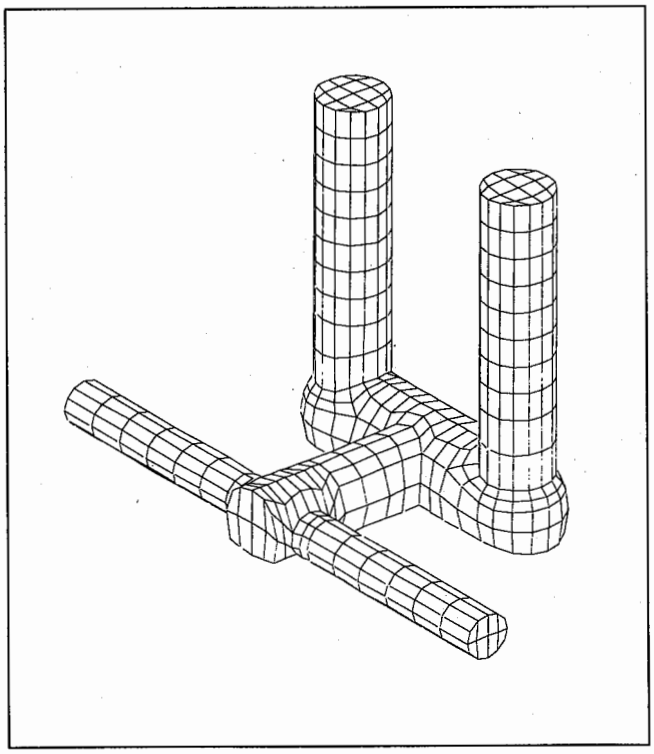

FIGURE 11. Finite element model for the liner of the combined personnel/utility shafts.

of the shafts and different shaft/adit configurations represented a significant design challenge. State-of-the-art analysis techniques included use of a high-speed minicomputer in conjunction with the three-dimensional software and a graphical pre- and post-processor used to develop and mesh complicated three-dimensional geometry. This combination of software allowed the development of new modeling techniques that accounted for stress introduction and distribution at each step of sequential excavation and support.

A typical model that was analyzed represented a 65-meter cube of rock. Computer programs were developed that automatically stepped the analysis software through the excavation sequence required to construct a shaft and adit. The problem was solved for each stage of construction. Beginning with an in-situ stress state, ground elements were removed from the model (simulating excavation) and the resulting stress state around the opening was calculated.

The liner elements were then added around the opening (in an unstressed state), more ground elements were removed as the excavation progressed and the problem was solved again. This time both stress distribution in the ground, as well as the liner, was calculated.

In analyzing the combined personnel/utility shaft configuration (see Figure 11), over 35 steps were used to excavate the rock and place sections of unstressed liner as the sequence progressed from the shaft top to the far end of the connecting adit. With over 5,000 elements and 24,000 degrees of freedom, this model was one of the largest non-linear analyses run in the United States. This technique called for computing stress and displacement at each excavation stage and each liner placement step (35 times). The huge amount of computations required the use of a high-speed minicomputer (with 256 megabytes of RAM and a six-gigabyte hard disk) usually running 12 to 24 hours to complete all 35 steps.

Since the analysis followed the excavation and construction sequence, the stress distribution during and after each excavation is much more representative than that resulting from traditional analysis. Since this technique accounted for the sharing of load between the liner and surrounding rock and load redistribution at each step of excavation, the stresses in the liner were generally lower than those predicted by a simple one-step finite element solution. This model justified the use of a thinner liner and translated into substantial savings in construction costs.

The step-by-step simulation of the sequential excavation and lining of the shafts was made possible through the extensive use of multi-point constraints (MPC) in the finite element models. The MPCs allowed the computer model to simulate the excavation of a section of ground, elastic/plastic reaction of the ground, and the placement of shotcrete lining in an unstressed state.

A typical shaft model was constructed by first creating a three-dimensional geometric outline of the rock to be excavated using the graphical processing software. Care was taken

in setting up the mesh to account for interfaces between the geologic strata and distances between the excavation and lining steps so that the construction was accurately modeled. The maximum depth of excavation before placing support was a function of depth and geometry. Since keeping the excavation below this maximum depth was critical to stability and safety, the contractor was limited by the contract specifications to a maximum distance as dictated by the analysis.

Once the rock to be excavated was meshed, the structural liner was added on the outside using a unique set of nodes separate from the nodes of the ground mesh, but corresponding one to one. Each node of the thin-shell-element structural liner was attached to the corresponding node of the ground using an MPC link. The three-dimensional analysis software allowed for the individual programming of the stiffness of each link for six degrees of freedom. The ground surrounding the shaft excavation was next meshed directly on top of the liner elements out to the boundary of the model. Figure 12 shows a cross section of the model used to analyze a magnet delivery shaft.

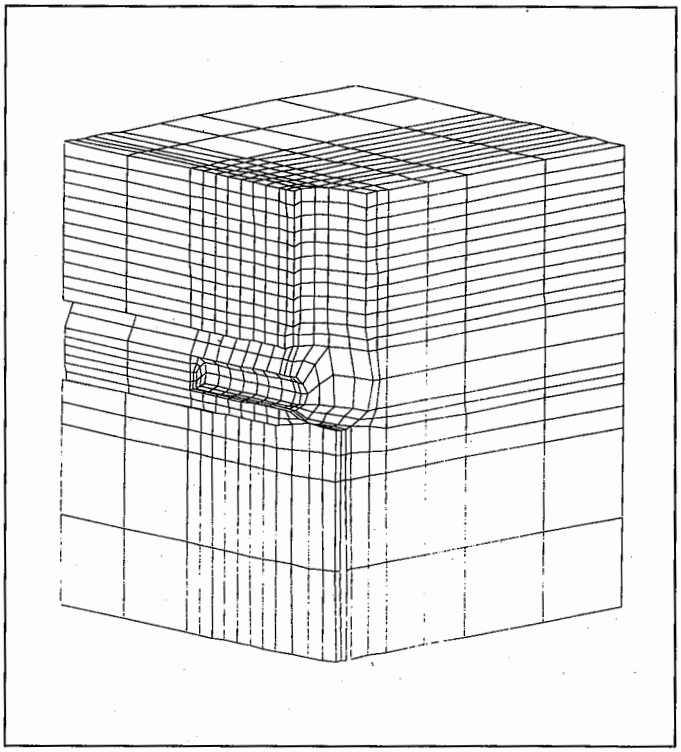

FIGURE 12. Cross section of the three-dimensional finite element model of the magnet delivery shaft.

The links permitted the ground surrounding the shaft to respond to the shaft excavation by deforming without transferring any load to the liner. Initially, the liner elements were turned off and the stiffness of the links was set to zero. In the first step of excavation, rock elements were removed and a solution calculated for the stress in the surrounding ground. Next, the links in the vicinity of the first excavation were turned on so that ground movement due to subsequent excavation steps would load the liner placed in the first step. This process was repeated through as many steps as needed to complete the excavation and lining of the shaft.

The process described above required the reprogramming of literally hundreds of links each time an excavation step was completed. This task, which could have required hours of manual manipulation, was automated by a custom program that modified the input file at the end of each step.

The results of the analysis were saved after each step, thereby allowing the ground and liner stress to be known at each step of the excavation. The graphical processing software was used to display stress and displacement (in three dimensions) using color bands directly on the model. The stress or displacement at each step could be played back by the processing software so that stress and displacement could be viewed as if it were a movie — showing how each excavation step affected the model. This mode of presentation was especially helpful in identifying critical steps. It also contributed to better overall understanding of how the ground responded to the excavation of complicated shaft and adit configurations (as shown in Figures 9 and 11).

Prior to the construction of the N15 magnet delivery shaft, extensive instrumentation was installed and deformation was monitored during excavation. The instrumentation data were used to check the validity of the computer mod-

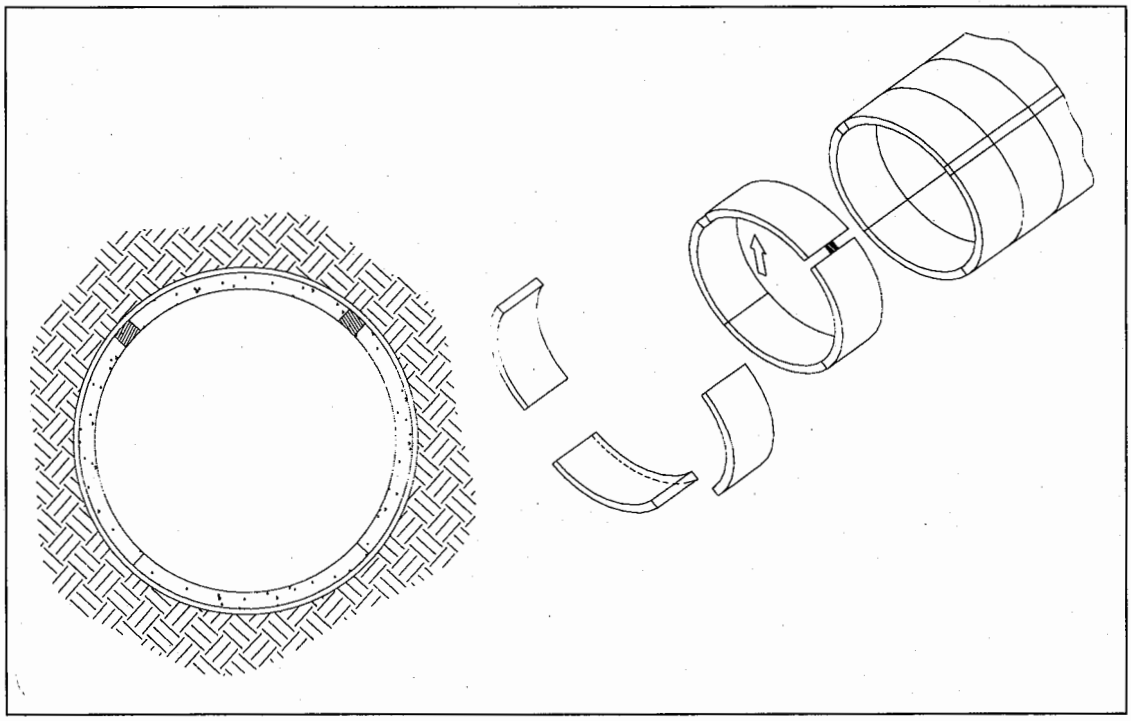

FIGURE 13. Precast segmental tunnel lining system.

els and calculated deformations. Field observation and computer simulations agreed within 20 percent, even before calibrating the computer model.

Structural Design

Tunnel Lining. Computer analysis of the tunnel in the AC verified early design assumptions that no lining would be required immediately or in the long term. Computer analyses of the tunnel in the EFS and TM also confirmed that a liner would be needed immediately following the excavation in the EFS and for the long term in the TM. A precast concrete four-segment liner was designed for both strata. The dimensions were kept constant to allow contractors to reuse forms, but concrete strength was varied to meet the specific needs of the geologic strata and tunnel depths.

The liner shown in Figure 13 is composed of four identical segments that were assembled inside the tail of the shield and expanded when clear of the shield. A closure strip was then filled with concrete. The liner thickness was 230 millimeters. The width of the liner, measured along the axis of tunnel, was 1.5 meters. The segments and closure strips formed a ring with an inside diameter of 4.25 meters.

The four segments used in forming the ring were approximately 86 degrees each. When assembled, there would be two gaps of approximately 300 millimeters on either side of the crown segment that would then be filled with concrete. The four segments were unbolted and ungasketed.

Structural materials were chosen based on considerations of strength, cost and quality control. The design of the reinforcement for the liner segments was based on individual Grade 60 bars. However, welded wire reinforcement could have been used if an equivalent area of steel were provided. The structural characteristics of the materials assumed for the tunnel lining were:

- Precast concrete strength of 55 MPa (EFS) and 40 MPa (TM)
- Young's modulus, concrete at 27,600 MPa (initial) and 13,800 MPa (long term)
- Cement grout strength of 15 MPa
- Yield strength of reinforcing at 400 MPa
- Young's modulus for steel at 200,000 MPa

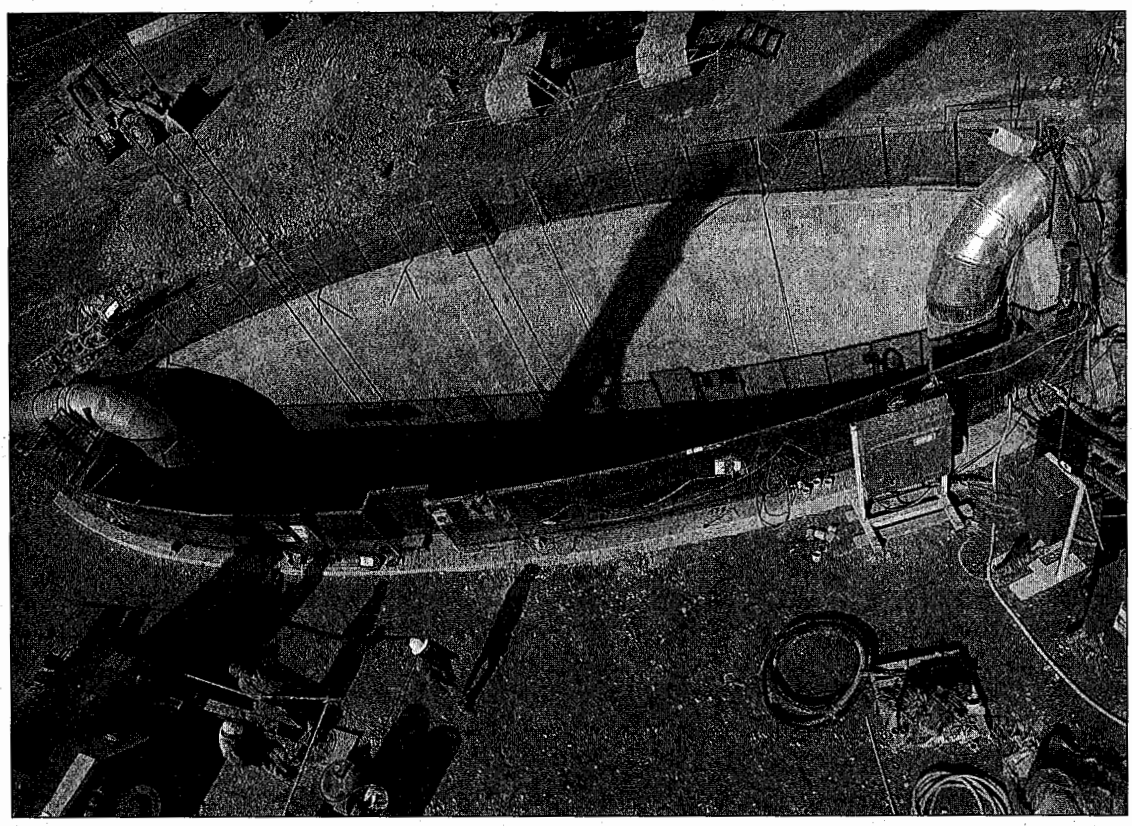

FIGURE 14. Surface collar of the magnet delivery shaft located at N40. (Photo courtesy of Benjamin Rodriguez.)

Concrete strengths of 55 MPa in the EFS and 40 MPa in the TM were used to resist long-term creep as well as jacking loads from the TBM. Some of the TBMs used to bore the tunnels were equipped with both hydraulic jacks and side grippers (inchworm machines). The reinforcement of the liners was dictated by handling loads, which included an impact factor of 200 percent to account for transportation from the precasting yard to the tunnel face. This factor was not required by the ACI code, but past experience has shown it to be a realistic value.

Shaft and Adit Lining. An initial lining of the circular shafts and adits was not expected to be required in the AC except at adit breakouts. The contractors were required provide a 100-millimeter layer of shotcrete for safety and to support softer layers of AC or bentonite seams.

The magnet delivery shafts and the circular shafts in the EFS and TM were initially supported with dowels and shotcrete. The thickness of the shotcrete varied with depth from 100 to 400 millimeters. Dowels were #9 or #11 bars, which would have been normally two to four meters long, cement grouted (full length). Dowels in the magnet delivery shaft were longer in order to extend beyond an assumed plastic zone adjacent to the large radius sides of these shafts. During excavation of the N15 magnet delivery shaft, which was excavated in the AC near the surface and bedded in EFS, 15-meter-long dowels were placed prior to excavation to keep the invert from heaving. Radially, five- to 10-meter-long dowels were used.

All shafts were designed with collars at the surface (see Figure 14). Those shafts bedded in the EFS also had an additional collar in the AC just above the EFS to aid in construction and to resist the anticipated differential horizontal pressures between the EFS and AC.

The final linings in circular shafts in the AC were to be cast-in-place concrete with minimum reinforcement and a strength of 35 MPa.

The purpose of these linings was to provide a smooth finish to mount utilities, staircases and ducts. Considering these requirements, a 250-millimeter wall thickness for shafts with a five- to eight-meter internal diameter in the AC was called for. These linings were designed to resist the long-term horizontal earth loads as well as vertical loads due to equipment and personnel.

Observations From Excavations

S30 Ventilation Shaft in the TM. The S30 shaft (see Figure 2) was 65 meters deep, entirely in the TM, and had an excavated diameter of six meters. The examination of the behavior of the TM was particularly important because the two large experimental halls on the east side of the collider ring were, for the most part, in the TM. Fifty-five-meter-high vertical walls, supported by tiebacks, were excavated in the TM, bottoming out in the AC. A number of vertical inclinometers 1.5 and 3.0 meters from the edge of the hole were installed to determine the lateral displacement due to the excavation and, hence, deduce the modulus of the material and the long-term creep (if any) of the marl. In addition, several piezometers were installed to record the groundwater response to the excavation.

The shaft was collared about 6.5 meters into the residual soil and weathered rock, using a circular secant-pile wall. Some six meters of additional weathered material were left before reaching unweathered TM. In this top zone, ground support was applied in the form of rock dowels 1.6 meters on center, and shotcrete. The remainder of the shaft was furnished with only eight centimeters of initial shotcrete ground support.

The shale stood up well, even when exposed unsupported for many hours at heights over three meters, and no signs of deterioration or distress were found. The maximum lateral displacement of the closest inclinometer casings at a 15-meter depth was between 0.25 and 0.40 centimeters, which is about three meters below the lowest rock dowels and near the top of the unweathered TM. At elevations below the lateral displacement was generally 0.08 to 0.20 centimeters. The deduced range of horizontal rock modulus is 700 to 1,400 Mpa.

Shafts Bottoming in Eagle Ford Shale. A number of shafts were constructed that penetrated through the AC and bottomed in the EFS (shafts at the N15 and N20 sites — see Figure 2). Shaft stations were excavated, and tunnel stubs were constructed that later met up with, or served as starter tunnels for, TBMs driving the collider tunnel.

These shafts, shaft stations and tunnel stubs were instrumented to determine the response of the ground to excavation and ground support activities. The instrumentation data and observations made were not trivial pursuits of scientific knowledge but served to calibrate the finite element models, to assess the safety and stability of these underground openings during construction, and to verify the magnitudes of loads or displacements that may be suffered by the permanent, cast-in-place concrete linings. The higher the rate of displacement occurring at the time the final lining is placed, the higher the loads eventually suffered by the final lining.

The construction of these underground openings was generally uneventful. For the most part, they were excavated using roadheaders and occasionally with compressed-air power tools. The excavation and support were carefully staged to minimize the exposure of the EFS before applying ground support. In the shafts, only about 1.5 meters of ground was excavated before placing first a layer of shotcrete, then lattice girders, followed by an additional layer of shotcrete completely embedding the lattice girders.

A pattern of rock dowels was also placed. The large, 10- by 20-meter oval magnet delivery shaft (N15) also was furnished with pre-installed deep rock dowels in the shaft bottom. These dowels were installed when the shaft excavation reached a level just above the AC/EFS interface (see Figure 6). The smaller ventilation and utility shafts were not furnished with bottom dowels.

The tunnel stubs were excavated in a piecewise fashion called the *sequential excavation and support method.* On the European continent a variation of this method is referred to as the *New Austrian Tunneling Method* (NATM). A 1.5-meter limit was put on the advance before ground support was applied, again in the form of shotcrete, lattice girders and dowels. This support was often combined with spiling,

which consists of dowels drilled and grouted in place in a fan pattern above the crown of the advancing tunnel.

The method succeeded in maintaining stability of the tunnel and the advancing face. Examination of the face rock in between advances disclosed the existence of latent, stress-induced fractures that were more or less parallel to the tunnel faces surface. These fractures permitted the removal of fairly large chunks of rock by hand. On two occasions, very large chunks of rock (over 100 kilograms, up to three meters in dimension) fell from the crown, bounded by a near-horizontal shear surface and near vertical shears.

Some of these shears may have been pre-existing, particularly near the AC/EFS interface, where the EFS was known to present a number of latent shears. Others were conditioned by the presence of horizontal bedding planes. The two falls occurred where excavation had, in fact, proceeded 2.5 to three meters ahead of the last installed lattice girder without shotcrete support or with only a thin coat of freshly placed shotcrete (not yet of sufficient strength). These events clearly demonstrated the value of treating the EFS with respect and minimizing the time and extent of unsupported exposure.

In the N15 (the magnet delivery shaft), heave measurements were performed at the shaft bottom using a very sensitive, pre-installed instrument that permitted the measurement of vertical displacement at one-meter intervals. Dowels were pre-placed in this floor to a depth of 20 meters below the floor. The dowels were evidently effective in reducing shaft floor displacements and maintaining elastic and stable conditions a short distance below the floor since measured heave was greatly below the amount predicted if no reinforcement had been used.

Conclusions

The size and scope of the SSC, as well as the variety of underground structures in the project, offered tremendous engineering challenges. The excavation and lining of complex underground structures in distinctly different geologic strata offered the opportunity to develop sophisticated analysis techniques.

Before analyses of the tunnels, shafts, adits and experimental halls could take place, an in-depth exploration of the in-situ geology as well as the calibration of the material properties of each of the strata were required. The instrumentation of excavations allowed the designers to both observe ground behavior during the various stages of construction and to calibrate their computer models used to predict design loads.

The three-dimensional sequential finite element rock-structure interaction analysis techniques (developed by the authors and calibrated during construction) provided valuable design information and insight into the development and distribution of stresses in the underground structures and surrounding rock at the various stages of construction and throughout the life of the structure.

The design of the underground structures, while following proven methods, also incorporated many state-of-the-art techniques. By accounting for the sharing of load between the rock and liner, a much thinner tunnel liner was justified. This liner was thinner than would have been dictated by traditional analysis and it provided a substantial construction cost saving. The cost savings over the entire project was far greater than the additional costs of analysis associated with the computer hardware and software required.

If used in conjunction with field observations, and with additional checks from classical analysis methods, the sequential interaction analysis technique can be a useful tool for economical and realistic design of underground structural systems.

ACKNOWLEDGMENTS — *The SSC was operated by Universities Research Association, Inc., for the United States Department of Energy under Contract DE-AC35-89ER40486. The PB/MK Team — a joint venture of Parsons Brinckerhoff Quade & Douglas, Inc., and Morrison Knudsen — was responsible for the analysis and design of underground structures and surface facilities. The two-dimensional finite difference program used for the project design analyses was Fast LaGrangian Analysis of Continua (FLAC), developed by Itasca Consulting Group of Minneapolis. The three-dimensional finite element program used for the pro-*

ject design was ABAQUS, developed by Hibbett Karlsen Sorenson of Providence, Rhode Island. PATRAN was the graphical pre- and post-processor used to develop and mesh the complicated three-dimensional geometry. The authors wish to acknowledge the cooperation of Tom Corry of Gilbert Construction Co. and thank Dr. Benjamin Rodriguez for the use of the construction photographs.

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A Framework for Modeling Pavement Distress & Performance Interactions

Developing a usable framework requires determining to what extent the various pavement characteristics influence the process of road surface deterioration.

SAMUEL OWUSU-ABABIO & JOHN COLLURA

Although a precise relationship between pavement distress and performance has not been definitively established, there is a consensus that the ability of a pavement to withstand traffic loads and serve the motoring public in a safe and efficient manner is adversely affected by observable surface distress. This distress, if untreated, ultimately becomes manifested as roughness, which interferes with proper surface drainage, causing water ponding on the surface, which in turn leads to negative impacts on the performance of the pavement and on vehicle safety. A review of research efforts also indicates that relatively little systematic modeling has been done to examine the interrelationship between distress and pavement performance despite its crucial importance.

Overview

The aging road infrastructure in the United States — coupled with the increasing pressure for governmental fiscal austerity and increasing demands for improved management and planning techniques — exemplifies to varying degrees the problems facing highway planners, financiers and engineers at national, state and local levels. At the core of addressing these problems is the need to assess the current and future condition of the highway network. This assessment, together with a coherent policy and set of maintenance standards, would aid in determining the level of service to be provided as well as in determining the funding required to support that level of service. Consequently, predicting pavement distress and performance has become the focus of many research studies in recent years as highway agencies acknowledge the need to upgrade their pavement management systems (PMS).[1]

A report on the feasibility of a PMS for the State of Connecticut identified the need to develop pavement performance prediction models that could be used for network-level budget optimization. Pursuant to this need, a long-term monitoring (LTM) program was initiated in 1984 by the Connecticut Department of Transportation (ConnDOT). The purpose of this 10-year study of in-service pavements with varying characteristics was to gather pertinent data in the form of pavement distress, skid resistance, roughness and maintenance history for 82 LTM sites. These sites consisted of 57 flexible, 11 rigid and 14 composite pavements. The flexible pavements were broken into three subcategories:

- Thirty-three sites that had overlays on liquid-treated pavements;
- Fourteen sites that had overlays on the original flexible pavement; and,
- Ten original flexible pavements having no overlays.

It was possible to develop a conceptual framework and analysis based on the data collected by ConnDOT between 1984 and 1989 on the 14 sites containing overlays on original full-depth pavements. The purpose of this analysis was two-fold:

- To develop and test a framework for modeling pavement distress and performance interactions; and,
- To identify areas needing improvement in the ConnDOT database system prior to developing more comprehensive performance models at the end of the LTM project.

The framework proposed herein uses pavement roughness (a quantitative measure of road surface distortions that result in an uncomfortable ride for the motorist) over time as a direct quantitative measure of performance, rather than a combined index such as the Present Serviceability Index (PSI). The use of roughness is based on results of several studies that concluded, in many instances, that roughness measurements alone may be sufficient in predicting pavement serviceability.[2-4] If a significant relationship does exist between distress and roughness, then a one-time expenditure on roughness measurement equipment may be all that a highway agency requires to determine road conditions and to monitor road performance. In that way, the agency can cut costs on extra equipment and other activities involved in the direct measurement of distress.

Studies have also shown that the cost of operating vehicles increases with pavement roughness. Therefore, the existence of a significant relationship between roughness and distress could provide a method to determine the relationship between distress and vehicle operating cost. At the local level, engineers are more familiar with the concept of distress rather than that of roughness.[5] Therefore, if the latter relationship (vehicle operating cost versus distress) is developed, it could provide a sound basis for engineers (particularly at the local level) in formulating their annual or periodic pavement maintenance needs that are presented before town budget committees.

A workable model should address the following questions:

- To what extent do various characteristics of a pavement — mix design properties, past distress levels, age, the environment (as represented by pavement regional location) and maintenance history (including the quantity of sealed cracks and the corresponding age of the crack sealants) — influence deterioration?
- Since cracking is the predominant distress that was observed on LTM sites in Connecticut, could it be used to represent all road surface distress?

Framework Development

General Concepts of Pavement Deterioration. The concept of pavement deterioration has been well documented by Paterson.[4] The mechanism of pavement deterioration is related to the physical or mechanical behavior of the pavement and its various components. Weather and traffic factors influence pavement deterioration. Repeated axle loadings induce various stresses and strains within pavement layers, resulting primarily in cracking. The combined action of weathering and oxidation also causes

bituminous surfaces to become brittle and, hence, more susceptible to cracking and disintegration, especially if the quality of the mix design and construction work were low. Once initiated, cracking levels (in terms of severity and extent) increase to a point where spalling occurs and potholes often develop. Open cracks on the surface and poorly designed and maintained drainage systems permit excess water to penetrate the pavement. This seepage accelerates the process of disintegration and reduces the shear strength of the subgrade and/or the subbase. These cumulative deformations throughout the pavement result in ruts in the wheel paths and in surface roughness.

The roughness of a pavement is, therefore, the result of a chain of distress mechanisms involving the combination of various modes of distress. A proper maintenance program can usually reduce the rate of deterioration, but certain forms of maintenance, such as patching, may increase roughness slightly. Roughness is thus viewed as a composite distress — comprising components of deformation due to traffic loading and rut depth variation, surface defects from spalled cracking, potholes, patching, and a combination of aging and environmental effects.

Distress & Performance. The concept of performance as a measure of highway deterioration has been widely analyzed and discussed by many researchers.[4,6-9] A combined index such as the PSI is a popular pavement performance analysis tool. Recent modeling techniques, however, are shifting from the combined index approach to a more objective measure such as pavement roughness. These techniques also adopt a more versatile approach in which major distress modes are individually modeled to better analyze and explain the relationship between distress and performance.

Pavement performance is usually defined as the variation in the level of service, or serviceability, provided to the pavement user over time. Distress, on the other hand, is a form of limiting pavement behavior characterized by perceptible evidence of physical deterioration.[10] Prediction models for performance can have either a probabilistic or deterministic basis.

Probabilistic models have utilized Markov's theory and have often been considered desirable because of their ability to recognize and accommodate uncertainty.[11] In spite of the elegant theory or structure of the Markov chain models, there is the problem of how to develop a transition matrix. The Markov process also depends only on the present state in predicting the future state. However, various studies have shown that other variables such as loading and age of pavement are also significant in predicting a future state. Markov-based models assume that transition probabilities are constant over time; *i.e.*, the Markov chain is assumed to be homogeneous. Since traffic loads generally increase over time, and maintenance methods also vary over time, this assumption may be unrealistic.

Deterministic analyses, as found in the literature, are based on functional performance (which relates objectively measured pavement behavior to the average user's opinion of serviceability), structural performance and damage assessment. For structural performance, features such as cracking, rutting and raveling are used to measure the physical condition of the pavement. Damage is typically quantified as a number ranging from 0 to 1. Typically calibrated using regression techniques, the deterministic model may be either based on empirical or mechanistic empirical correlations.

The use of roughness as the primary measure of performance (*i.e.*, serviceability over time rather than a combined index such as PSI) seems more useful since the user's perception is dominated by riding comfort, which is usually estimated by roughness.[3] Hence, by relating roughness to different combinations of various amounts of distress measured during a pavement's life, the serviceability-age profile or performance of a pavement can be realistically related to distress.

As outlined by Paterson, there are also major problems with using a summary statistic such as PSI as a performance parameter:[4]

- Different types of maintenance are appropriate for different levels of each distress type.
- The relative seriousness of different defects varies with the pavement type, envi-

ronment, the rate of deterioration and the maintenance program. (For example, the relative weightings developed for asphalt pavements in the wet-freezing climate of the Illinois region as a result of the AASHO Road Test are not necessarily applicable to pavements with thin surfacings or composite pavements, nor are they suited to pavements in a dry non-freezing climate.)
- Each distress type evolves at different rates in different pavement types and under different traffic and environmental conditions.

Thus, modeling the performance by using PSI alone requires determining the average amount of distress from the many different combinations of these types of distress. This method could yield results that have wide variances that, in turn, may suppress the very effects that are of interest.

Framework Assumptions. Based on the preceding discussion, the following concepts serve as the assumptions that can be employed in developing a model for relating pavement performance and distress:

- The future characteristics of a pavement depend on how those characteristics have performed in the past (*i.e.*, pavement characteristics are recursive in form). Hence, the level or amount of a specific type of distress has a direct relationship to the level or amount of distress that was last observed.
- Pavement aging leads to increasing amounts of distress.
- The amount of distress observed depends on the previous traffic levels (expressed in terms of equivalent single-axle loadings).
- The amount of distress observed depends on the maintenance history of the pavement (for example, pavement age as well as the quantity of sealed cracks).
- Cracking may be used as a surrogate for all distresses in representing road surface roughness (since it was the most significant observed distress on LTM sites in Connecticut).

The framework that is depicted in Figure 1 illustrates the complex relationships that exist among pavement deterioration factors (or inputs), the deterioration process, serviceability and performance. Models for various distress types — for example, cracking or rutting — represent the deterioration process. These models use the deterioration factors — such as weather, loading, or construction and maintenance practices — as inputs to predict distress. For this framework, pavement roughness is considered to be the primary indication of distress. Thus, roughness is viewed in terms of the road surface distortions that contribute to an undesirable or uncomfortable ride.

Rough pavements, when untreated, generate increased dynamic loads that hasten the deterioration process. This relationship suggests that beyond a certain level of distress, there is a two-way causation between roughness and distress. However, since cracking was the only form of distress employed in this framework, the impact of increased dynamic loading was assumed to be insignificant. If far greater levels of other types of distress such as potholes and corrugations were encountered, modifications would certainly have to be made to the framework. Some forms of maintenance (for example, patching) may increase roughness slightly even though these practices are intended to reduce pavement deterioration. Over time and/or usage, the serviceability-time or usage profile portrays the general performance of the pavement.

Framework Examination

Using the set of assumptions presented earlier, pertinent data were required to examine the efficacy of the framework shown in Figure 1. The database used for this task included a time series data set on 14 segments of flexible pavements gathered between 1984 and 1989 by ConnDOT as part of its long-term pavement monitoring study. The database consisted of the average annual daily traffic (AADT), pavement cross section, core characteristics, pavement age since the last overlay, South Dakota Road Profiler roughness measurements, sealed and open cracking, patching, skid resistance and environmental information (as repre-

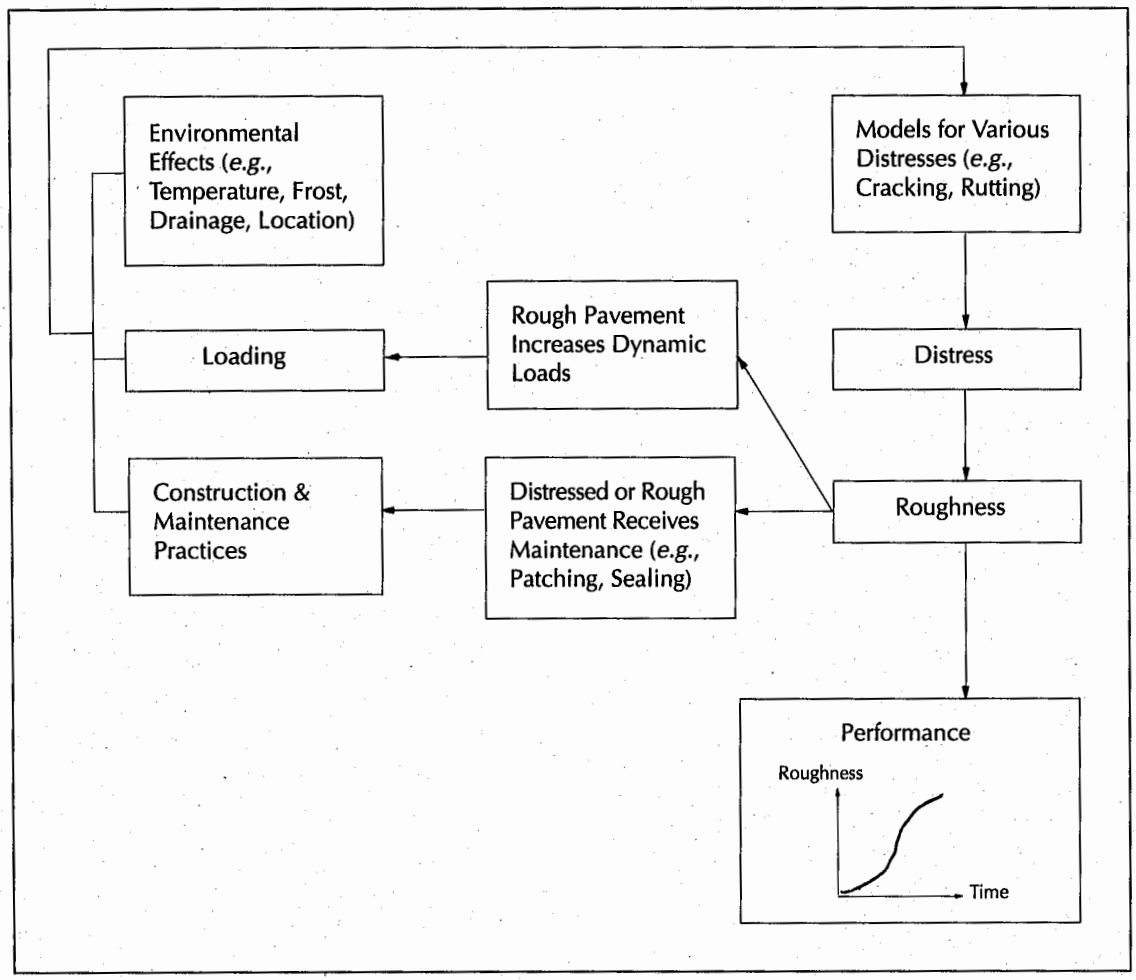

FIGURE 1. A framework for relating pavement distress to performance.

sented by pavement regional location). The main focus of this study was on cracking, since significant amounts of other types of distress were not encountered on the ConnDOT LTM sites. Each of the 14 segments was 1,000 feet long and contained overlays on the original flexible pavements. The age of these overlays ranged in age from three to 15 years.

Data Analysis

Analysis of the data consisted of two phases: a preliminary phase and a model-building phase. The former used cross classification, correlation and regression analyses techniques to select key variables for the model-building phase. Additional regression analyses techniques were performed during the model-building phase on the variables that were assumed to influence pavement deterioration.

The first step involved in the preliminary analysis was to compute for each pavement the structural numbers from the core information, the equivalent single-axle loadings (ESAL) from the AADT data and extract from maintenance records the amount of sealed cracks (as well as the corresponding age of the seal at any time within the observation period).

The next step was to subject the entire data set to further analysis using cross classification, scatter plots and correlation techniques. A series of regular and non-linear regression analyses was then performed to examine whether significant relationships existed among the variables. A fundamental assumption was

TABLE 1.
Variables & Functions for Regression Results

LC_t = Open longitudinal cracking in year t (ft/100ft)
TC_t = Open transverse cracking in year t (ft/100ft)
$TOTC_t$ = Combined or total cracking in year t (ft/100ft)
RN_t = Roughness in year t (inch/mile)
TS_t = Sealed transverse cracking remaining in year t (ft/100ft)
$TOTS_t$ = Combined or total sealed cracking remaining in year t (ft/100ft)
PAGE = Pavement age since the last overlay (years)
R^2 = Coefficient of determination
SAGE = Age of sealed cracks (years)
SN = Pavement structural number
VTYP = Dummy variable for equipment used for roughness measurements

General Functions for Dependent Variables — Pavements With Sealing History

$TOTS_t = f(PAGE, SAGE)$
$TOTC_t = f(TOTC_{t-1}, PAGE, SN, TOTS_{t-1}, SAGE)$
$TS_t = f(PAGE, SAGE)$
$TC_t = f(TC_{t-1}, PAGE, SN, TS_{t-1}, SAGE)$
$LC_t = f(LC_{t-1}, PAGE, SN, SAGE)$
$RN_t = f(RN_{t-1}, SAGE, TOTC_t)$
$RN_t = f(RN_{t-1}, SAGE, TC_t, LC_t)$

General Functions for Dependent Variables — Pavements Without Sealing History

$TOTC_t = f(TOTC_{t-1})$
$TC_t = f(TC_{t-1})$
$LC_t = f(LC_{t-1}, ESAL_{t-1}, PAGE)$
$RN_t = f(VTYP, SN, RN_{t-1})$

made that maintenance, in the form of crack sealing, can retard pavement deterioration. Therefore, stratification of the data during classification analysis was necessary so that trends could be observed separately for segments with and without crack sealing history. (The term *history* is used here to mean whether sealing has been done on the pavement in previous years.)

The correlation analysis step involved selecting the appropriate form of the variables that could be used to explain the relationships among traffic, the environment, distress and serviceability (as measured in terms of roughness at a specific time). A correlation matrix was set up to determine whether there was a relationship between pairs of variables. A test of this proposition involved examining the data to find out if they support such a proposition. This matrix was employed to examine the strength of association between a dependent variable and an independent variable.

While a correlation analysis provides an assessment of the strength of the relationships among the variables, a regression analysis on the other hand permits drawing conclusions about the functional relationships existing among the variables. The regression analysis option of the statistitical analysis software (SAS) employed in the analysis was used to perform all analyses pertaining to regression. The SAS regression analysis option was performed only for cases with no missing data for any of the variables selected for analysis. The regression equations were evaluated using the following criteria:

- Test of significance of regression (F-test) to assess the overall significance of fitting the regression equation.
- Test of significance of each variable (t-test) to determine how important any one term is in the regression equation after all other terms have been included.

Framework Model Building

The model-building phase for the proposed framework consisted of selecting the key variables that could be used to explain the mechanism of distress formation and the interaction between distress, performance, traffic and the environment (a dummy variable represented by pavement regional location). The pave-

TABLE 2.
Model Parameters for Pavements That Have Not Experienced Any Crack Sealing

Dependent Variable	Independent Variable	Variable Coefficient	t-Value	Constant Term	Variable Exponent	Model R^2
$TOTC_t$	$TOTC_{t-1}$	1.28	20.61	—	—	0.953
TC_t	TC_{t-1}	1.10	25.34	—	—	0.968
LC_t	LC_{t-1} $ESAL_{t-1}$	1.13 5.16×10^{-5}	8.08 2.15	—	—	0.953
RN_t	RN_{t-1} SN	1.10 54.09	2.40 2.31	—	0.75 -2.00	0.837

ments under study fell within three regional locations: the shoreline area close to Long Island Sound, the Connecticut River valley, and a hilly area (which tends to have a much cooler climate compared to the other two locations).

The dependent variables for each of the two broad categories (cracks with and without a sealing history) included: roughness, transverse cracking, longitudinal cracking and combined cracking (which is the sum of all cracking forms observed, predominantly longitudinal and transverse cracking with very few cases — less than one percent — of alligator cracking). The independent variables that would explain the variation were selected based on the results of the correlation and regression analyses. The only environmental variable considered in the regression analyses — *i.e.*, the pavement regional location — was not statistically significant and, therefore, was dropped from further consideration. The selected variables are defined in Table 1.

Using the generalized functional forms also presented in Table 1, the final values obtained as a result of further regression analyses are summarized in Tables 2 and 3. (A variable appearing with subscript *t-1* denotes the quantity of the variable in the previous year.) Only independent variables that were statistically significant were included in the final results. The

TABLE 3.
Model Parameters for Pavements That Have Experienced Crack Sealing

Dependent Variable	Independent Variable	Variable Coefficient	t-Value	Constant Term	Variable Exponent	Model R^2
$Loge TOTS_t$	PSAGE*	-0.04	-3.85	3.52	—	0.220
$Loge TS_t$	PSAGE	-0.04	-5.31	2.99	—	0.357
$TOTC_t$	TOTCS** PSAGE SN	0.66 98.14 -34.79	9.88 4.16 -4.73	—	0.10	0.921
TC_t	TSCT*** PSAGE SN	0.53 24.33 -7.96	6.43 3.38 -3.79	—	0.10	0.918
LC_t	LCB§ PSAGE SN	1.16 47.07 -14.56	8.80 4.52 -3.88	—	—	0.903
$Log_{10}RN_t$	$Log10RN_{t-1}$ $Log_{10}SAGE$	0.52 0.41	4.37 2.96	0.5	—	0.546

* PSAGE = PAGE × SAGE
** $TOTCS = TOTC_{t-1} - 186.21/(TOTS_{t-1})^{0.23}$
*** $TSTC = 6.93 \times (TC_{t-1})^{0.51} + 60.26/(TS_{t-1})^{0.24}$
§ $LCB = 6.92 \times (LC_{t-1})^{0.56}$

TABLE 4.
Summary of Actual & Predicted Cracking & Roughness Data for 1990

LTM Site	Longitudinal Cracking			Transverse Cracking			Total Cracking			Roughness		
	Actual	Predicted	Error (%)	Actual	Predicted	Error (%)	Actual	Predicted	Error (%)	Actual	Predicted	Error (%)
F1	105	90	14	61	42	31	166	119	28	28.8	12.4	57
F2	160	132	18	66	49	26	382	326	15	8.3	14.3	72
F3	41	87	111	56	59	5	217	156	28	23.1	31.0	34
F4	102	121	18	33	35	6	243	136	43	12.4	23.7	91
F5	11	45	303	14	37	158	25	87	248	2.1	19.6	833
F6	141	126	11	52	39	25	193	154	20	3.6	10.5	192
F7	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
F8	228	116	49	34	60	76	262	162	38	42.5	29.0	32
F9	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
F10	NA	NA	NA	1	2	100	1	2	100	4.2	7.2	71
F11	43	77	79	27	44	63	70	113	61	27.7	38.5	39
F12	65	77	18	83	57	31	148	143	3	14.6	41.4	184
F13	154	136	12	54	52	4	208	190	9	36.6	41.3	13
F14	160	216	35	55	61	11	215	276	28	12.5	10.1	19
Mean*			37			26			27			73
Number of Observ.		10			11			11			11	

*Mean prediction errors computed excludes LTM site F5.

results in Tables 2 and 3 indicate that future cracking levels are highly dependent on past cracking levels regardless whether the pavement has received crack sealing or not. Table 2 further indicates that for flexible pavements that have not received any form of crack sealing, the previous cracking levels alone may be sufficient in predicting future cracking levels. This conclusion is supported by the high R^2 values associated with the results from the various cracking forms.

Framework Model Testing

The robustness of the model that was developed was tested by predicting the 1990 characteristics for the LTM sites. The 1990 data were not part of the data set used for the model calibration. Table 4 shows a summary of the 1990 actual and predicted cracking and roughness data for the 14 sites. Characteristics for sites F10 and F14 were predicted using the model for pavements without a crack sealing history. Those sites with a crack sealing history (with the exception of F9) were predicted using the model for pavements with a crack sealing history. In Table 4 the predicted values were rounded to whole numbers; however, errors associated with prediction were computed based on the as-predicted values. Table 4 also presents predictions for only 12 out of the 14 sites since descrepancies existed in the 1990 data for sites F7 and F9.

It can be seen from the data in Table 4 that a majority of the errors associated with the predicted roughness values are extremely high. The mean error of roughness prediction is approximately 73 percent (not including site F5). This number goes up to 136 percent if site F5 is included (which makes the model less attractive). Consider the three sites with roughness prediction errors greater than 100 percent: sites F5, F6 and F12. Their respective 1989 roughness values were 14.1, 4.2 and 34.2 inch/mile, respectively. The expected trend for 1990 was that, without any major improvements, these numbers should have been at least equal to their previous values. However, the actual 1990 values (see Table 4) were much lower than

expected. The large errors in the roughness category might be due to such factors as the inability of driver to drive in the previous year's wheel paths or errors in the devices used to measure the roughness values. The mean predicted error ranged from 26 percent for transverse cracking to 37 percent for longitudinal cracking (both excluding site F5).

Limitations

The limitations of this type of analysis may stem from data acquisition problems rather than from any conceptual flaws. The development of an appropriate prediction model for pavement network optimization would depend to a large degree on the reliability of the acquired data. Data reliability is important and cannot be overlooked in subsequent statistical analysis.

A significant limitation faced in this study was the unavailability of adequate traffic loading data for a majority of the road segments. Therefore, these data had to be estimated using AADT and the assumed growth rates for various truck types. The assumed growth rates were obtained from other road segments that were believed to have similar characteristics. It was expected that traffic loading would create a positive impact on levels of cracking and pavement roughness, but this was not the case. The element of loading could, therefore, not be adequately represented in the final models.

The preliminary analysis indicated that there was an extremely weak relationship between roughness (a dependent variable) and cracking and the type of vehicle used to make the roughness measurement (independent variables) compared to other independent variables such as the age of the crack sealants and the pavement structural number. This anomaly could be due to a number of factors:

1. Distress ratings on the ConnDOT LTM sites were conducted on a specific 100-foot sample unit within a 1,000-foot road segment. A summary of the roughness values corresponding to the 100-foot sample unit was unavailable at the time of the study, but it was nevertheless decided to attempt to correlate roughness with cracking.

2. In 1984 ConnDOT made special runs on LTM sites to obtain roughness values and to precisely demarcate start and end points. However, beginning in 1985, all roughness data were obtained using computer files for normal photolog inventories. (A photolog inventory is conducted using a special type of van equipped with facilities to collect photographic images of the roadway and roughness measurements from an axle-mounted accelerometer.) It is suspected that since 1985 there could be deviations of up to 0.03 mile between the position of the vehicle and the actual log mileage. This discrepancy suggests that the roughness summary may not completely cover the actual test segment.

3. A number of characteristics for the roughness measurement vehicles have changed over the years. For example, between 1984 and 1986, the measurement vans had a tire and a wheel size of 19 inches. This dimension was reduced to 16.5 inches beginning in 1987 when radial tires were mounted. Even though ConnDOT used the same van for roughness measurements, each year certain maintenance procedures could produce variations outside the normal expected trend. ConnDOT readjusted the vans for each test season using potentiometers. The vans were driven over a couple of roads and not over the same set of calibration bumps. These vans and accompanying equipment then were readjusted on the basis of potentiometer output that may be widely different from the previous year's.

4. Reliable data depends on the ability of the driver of the van to repeat the previous year's lateral path through the test site. This practice ensures that roughness measurements are made over the same spots each year. However, it is a difficult task even for the same driver to take the exact same path through a road segment. If the driver of the van varies each test season, the lateral path will most certainly vary.

5. The components of the vehicle may degrade over time. Shocks, springs and tires wear and are replaced. Equipment failures require the repair and replacement of circuit boards and electronic devices.

6. Even if data acquisition procedures for all other variables could be held constant, site condition data can still vary if a wide range of posted speed limits exists for these segments. Roughness has been found to be speed dependent for response-type roughness measuring devices. Therefore, two segments of "similar" surface characteristics, for example, would be likely to have different roughness values due to differences in posted speed limits.

The model testing results indicated that cracking predictions for some sites, particularly F5, were two to four times higher than the cracks actually observed. These divergences make the model less attractive. However, this seeming unreliability could be due to the fact that the 1990 data were collected by a survey crew that was different from the usual crew that had performed the survey for the past six years. Even though standard procedures existed for conducting the survey, measurement errors might have occurred because of unfamiliarity with proper measurement methods.

Conclusions & Recommendations

In developing this conceptual framework for relating pavement distress to performance, pavement roughness over time was used as a direct quantitative measure of performance. This approach was based on the results of a number of studies. Since distress ultimately manifests itself as pavement surface roughness, the framework suggests that to better understand the interrelationship between distress and performance, various types of distress should be modeled as independent modes of distress and then their relationship with roughness examined.

The proposed framework was tested using data from 14 pavement segments located in Connecticut. Various forms of cracking — including predominantly longitudinal and transverse cracking, with less than one percent alligator cracking — comprised the only type of distress that was modeled in testing the framework since it was the only significant visible distress occurring on the highway pavements monitored.

Based on the results of this study, the data do not adequately support the contention of a relationship between cracking and roughness. The suitability of the proposed framework could, therefore, not be fully assessed primarily because of data limitations. Further investigation is recommended. The data, however, support the following observations:

- The progression of cracking is primarily a function of previous cracking levels for pavements that have experienced some form of sealing (see Table 3). In addition, the previous level of cracking alone may be sufficient for predicting the future cracking for pavements that have experienced no sealing in the past (see Table 2). The future roughness of a pavement is also found to be primarily dependent on its current and past roughness (see Tables 2 and 3). These observations, therefore, do support the hypothesis that the future characteristics of a pavement depend on the history of those same characteristics.
- Although it is widely claimed that the age of a pavement will have a significant impact on deterioration, this analysis shows that the age alone may not be treated in isolation for pavements with a crack sealing history, because it is the interaction between the pavement age and the age of sealed cracks that influences deterioration.

The following recommendations regarding data collection and analysis should be considered for future modeling of pavement behavior:

- Having to adjust roughness measurements to posted speed limits suggests that segments with the "same" surface characteristics are likely to have different roughness values due to differences in posted speeds. For such situations, maintenance decisions based on roughness data are likely to trigger a wide range of maintenance actions that could be uneconomical. Therefore, it would be advisable to develop procedures for measuring roughness at some standard speed and restrict applying it to situations where obscure pavement geometrics, pedestrian and vehicular traffic would make it

impossible to operate at the standard speed.

- As far as cracking is concerned, the mechanisms that induce roughness are the effects of spalling and unevenness generated across cracked blocks of surfaces and the birdbath-type depressions that often result from localized deformation in the base as a result of surface cracking. Hence, for a more objective examination of the relationship between cracking and roughness, it might be useful to consider only medium to severe cracking in future modeling regarding the relationship between the two variables.
- The process of collecting distress data based on the definition of the term *cracking* as the combination of open and sealed cracks should be abandoned. Sealed cracks are assumed to prevent water from penetrating the pavement, consequently retarding the deterioration process. Therefore, to model cracking it is recommended that the term *cracking* be redefined to differentiate among open and sealed cracks. However, if the combined definition is adhered to, then future modeling of cracking should incorporate a separate model for sealed cracks, which can be used to extract the amount of sealed cracking embedded in the combined model.
- Further preliminary results of the ConnDOT study indicate that there is a positive relationship between cracking and roughness. However, the results of the significance tests suggest that this relationship is extremely weak. Therefore, the central question of whether cracking can be used as a surrogate for all distress in representing roughness cannot be definitively answered based on the results of this research. Further investigation is required. ConnDOT could extract from its mileage records summaries of roughness data corresponding to the 100-foot units over which distress was rated. These data might provide a better basis to examine this relationship.
- Longitudinal and transverse cracking formed the predominant type of distress analyzed in this study. Both types of cracking result from an environmental fatigue process that is determined largely by material characteristics and the temperature regime. It would make better sense to employ pavement temperature as a more specific environmental variable in future modeling of these cracking types rather than the regional location dummy variable considered here. Longitudinal cracking, however, also may be due to loading if these cracks are found in the wheel path.[12] As the level of alligator cracking and other manifestations of distress such as rutting becomes pronounced, the effect of loading could be more significant and would require efforts to develop appropriate loading data for consideration in future modeling.

ACKNOWLEDGEMENTS — *Sincere thanks go to Professors Paul Shuldiner and James Kindahl of the University of Massachusetts. The participation of the Research Division of ConnDOT deserves recognition. The exceptional and constructive comments offered by Donald Larsen, Transportation Supervising Engineer, and Anne-Marie McDonnell, Transportation Engineer, in the ConnDOT Research Division, contributed to the overall success of this study. The efforts of Jeff Scully, Transportation Engineer in the Research Division, in gathering pertinent data are also greatly appreciated.*

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Transportation Planning Policy for the 21st Century

Adopting new technologies and fiscal policies along with changes in how the federal government works with private and other governmental sectors are key to meeting future challenges.

MORTIMER L. DOWNEY

The United States has the largest, safest and best transportation system in the world. It is a good reason why a continent-sized nation bounded by vast oceans and split by mountain ranges and rivers has been able to move people and goods so efficiently. Transportation has been critical to this country's economic security, ensuring a high quality of life.

Indeed, the U.S. transportation system works so well that most of its users take it for granted. Little thought is given to how well it is working. Only when problems occur — when congestion slows rush-hour traffic or a flight is stacked up in a holding pattern — does the traveling public pay attention to transportation.

In these instances, it is all too apparent that a slow-moving disaster is approaching the transportation system in the U.S. that could be as potentially disruptive as a flood or an earthquake. This slow-moving disaster is actually a composite of three factors:

- The nationwide transportation infrastructure deficit that is estimated (by some) in the hundreds of billions of dollars;
- A rapidly growing demand for travel services that is outstripping capacity; and,
- An increasing need for efficiency (for example, 50 percent of U.S. manufacturing businesses rely on prompt deliveries for "just-in-time" operation).

When confronted by such challenges, the strategy for more than a generation has been to look for public funds — usually in the form of federal grants — to build more highways, airports and mass transit systems.

However, that era is at an end. The need to reduce the national budget deficit is going to limit future federal spending. Therefore, new strategies for funding and implementing work to maintain and improve our transportation infrastructure have to be developed.

Not only limited funding but also the sheer cost of new construction — especially in urban

areas — restricts the ability of the federal government to undertake many of the types of projects that characterized the Interstate highway era that burgeoned after World War II. No government — national, state or local — can afford to build too many new roads when they cost $127 million per mile (as the Century Freeway in Los Angeles did).

Money is not the only limiting factor. Other legitimate concerns, such as reducing negative impacts on the environment, place additional limitations on developing new projects and on how projects are constructed.

There is no single solution to the problems that the United States faces in maintaining and upgrading its transportation system. However, a variety of strategies and solutions that are complementary can help this nation meet its transportation needs in the 21st century.

A primary part of this matrix of strategies is to adopt a change of attitude in how the government responds to problems with the national transportation infrastructure. There has to be a shift from expanding and building onto the transportation system to managing the existing system.

Of course, while construction may represent less of a part of the total solution as it has been, the U.S. is still going to have to build as well as re-build key works. Doing so in an era of tight budget limits means doing things smarter — developing, implementing and constructing these projects in a more efficient manner than the government and public have been accustomed.

There are three new strategies that show great promise for working with the new limitations:

- Intelligent Transportation Systems (ITSs);
- Innovative financing practices; and,
- Improving the ways in which the federal government works with its partners in state and local governments as well as the private sector.

Intelligent Transportation Systems

ITS represents a more "wholistic" view of transportation. ITS is what used to be called Intelligent Vehicle-Highway Systems (IVHS) until the transportation profession realized that the focus should not be just on vehicles and highways, but on the entire complex of transportation systems — road, rail, air and water.

Many people think of ITS as only automated highways. They also think ITS falls within the realm of science fiction. But, prototypes of these systems have already been implemented. Over the past couple years, a sound foundation for the wider deployment of ITS has been instituted. Nearly 70 operational tests of ITS have been launched, providing much useful information on what aspects are possible to deploy now, with today's technology, and what aspects still need further development.

In addition, the first phase of designing a national architecture for ITS has been completed. This architecture serves as the equivalent of a biological genetics system — this architecture will be the DNA that guides the full deployment of ITS. The process of creating an architecture also has helped define a core ITS infrastructure composed of communications and information links. These links are essential to even the most basic ITS services and products.

A key to success in deploying ITS is mapping out how this architecture will be realized. To achieve that end, the new ITS National Program Plan charts an implementation course that details, with broad and specific clarity, how this architecture will be put into practice. Over the next year, many of the plan's elements will come to fruition.

For example, the planning process has been completed for each of the four ITS Priority Corridors — in California, Houston, the Midwest and the Northeast. These corridors have received special Intermodal Surface Transportation Efficiency Act (ISTEA) funding to develop integrated, regional ITS programs. They are launching real-world, real-time traffic management and traveler information services. The SmarTraveler program, operating in Boston, is one such program. With SmarTraveler, a driver can dial a phone number that connects to a central database that provides real-time information on traffic jams and alternate route suggestions.

By 1996, 100 Motor Carrier Safety Assistance Program (MCSAP) sites will have completed

the second phase of plans for an Internet-type commercial vehicle data exchange system. Working with this system, for example, police or regulators in California would be able to access the records of a truck registered in New York in *real time*. This capability will reduce, and possibly end for many carriers, the delays they now face at each state border they cross. MCSAP also could streamline and speed up routine motor carrier regulation, letting states focus on other safety and law-enforcement activities.

This program serves as a model for a new way to implement and enforce regulations, one that takes full advantage of what the latest developments in information technology can offer.

Most significantly, the work done over the last few years in developing prototype ITS programs has provided better understanding of, and insights to, the likely paths that ITS development will take. Deployment will begin with a core infrastructure comprised of communications and information systems. That type of infrastructure will allow public agencies to manage traffic more effectively, provide real-time information on travel and mass transit services, reduce the personnel needed to monitor truck safety, perform vehicle registration checks and collect fuel taxes.

This core infrastructure will enable a number of private products and services to come to market, or at least to increase the appeal of some that are already commercially available (and possibly decrease their cost). These products and services include Mayday devices and in-vehicle navigation and travel information systems — both of which are now being offered on model year 1996 cars.

A good deal of this core infrastructure is deployable now. The next phase of ITS deployment extends over the coming decade. During that period ITS will grow substantially, both in the number of services and users, as well as in their sophistication. Some of the products that will come to market over that period should include advanced traffic control systems and expanded in-vehicle route guidance systems. Programs that utilize intermodal "smart cards" will most probably be initiated. These smart cards will enable regulatory authorities to employ such market-based measures as congestion pricing and user fees to furnish a practical solution for traffic congestion.

The next phase represents truly entering the 21st century. A number of sophisticated technologies will be ready for day-to-day usage. Foremost among these new developments would be those to improve driver performance and avoid crashes: intelligent cruise controls, assisted steering and assisted braking.

Finally, about 20 or so years from now, the first fully Automated Highway Systems will go into operation. Initially, these systems will be used in special applications, such as on congested bridges or tunnels in New York City or on western highways where the long distances make driver fatigue an issue.

The federal government will support the implementation of ITS by serving as a *catalyst* — as a source of seed money and of expertise to foster deployment. In this way, the federal government will ensure the formation of institutions and systems needed for future growth.

The federal government will also provide leadership in standard setting by forging consensus on technological standards. The federal government is in the best position to help develop, promote and institute national and, possibly, international standards. The goals of these standards are to reduce entrepreneurial risk and create a stable, common ground to foster compatibility for consumers and companies for the interstate, and internation, movement of people and goods.

In addition, the federal government will serve as a facilitator and promoter of technology development. Efforts in this regard will be toward sponsoring research and building new alliances with U.S. industry.

Currently, the development of ITS is at an exciting juncture — on the verge of making systems that will benefit the average person — and the federal government is taking a leading role in this process.

Innovative Financing

In January 1995, President Clinton announced plans for the Partnership for Transportation Investment (PTI). This nearly $2 billion program will jump start 35 new transportation projects in 21 states. PTI reflects a change in the federal government's view of its role in fund-

ing transportation projects. The program reinvents the way that the federal Department of Transportation (DOT) does business. It gives states and localities greater flexibility, more authority and the ability to leverage private capital for transportation projects.

The federal government has traditionally supported transportation projects through matching grants to the states. However, this process has become far too rigid, inflexible and costly. For example, it has been almost impossible for states and localities to enlist the help of businesses and community organizations to get needed projects moving.

President Clinton, who struggled with such outdated federal rules when he served as Governor of Arkansas, issued an executive order to promote better project decision-making and different ways of financing infrastructure projects. Acting on this presidential directive, DOT asked the states to propose projects that they wanted to build, but had not been able to undertake because of restrictive federal rules. DOT assured the states that the federal government would use maximum flexibility within the law to see that valid projects would be financed and built. And that's exactly what we are doing.

For example, DOT is now allowing states to use private dollars as a substitute for the states' share of matching funds. DOT is also using federal funds to set up local revolving loan programs or to serve as collateral for state and local lines of credit. In addition, DOT is changing reimbursement rules so that states can start collecting federal funds while they are building, instead of having to accumulate the full federal share before breaking ground.

The benefits from these changes are expected to be impressive. These new practices will provide the impetus for new projects that will reduce traffic congestion, provide faster freight shipping and deliver better air quality in a more timely manner than would have been possible with the old way of doing business. There will be many immediate economic benefits gained from the more efficient movement of people and goods, from the strengthening of regional economies and from generating tens of thousands of new jobs.

These policies and programs will ultimately save taxpayers money. These savings accrue because being able to start construction sooner will avoid increased costs due to inflation and the interest burden on loans or bonds will be reduced. Projects could realize a savings of 15 percent per year from accelerated construction alone.

This common-sense approach to cutting red tape also will attract more private resources, providing a means to leverage them and, thus, freeing scarce public resources for other investments.

More projects using such innovative financing methods will be announced in the future. These projects will not only be targeted for highways, but also for airports, seaports and mass transit systems as well as all the links between them.

DOT is asking the states to continue seeking out and developing new financial policies in all aspects of project funding. One example is Massachusetts' proposed "wrap-up" insurance for management and design work for the Central Artery/Third Harbor Tunnel Project's many consultants. This proposal, which the Federal Highway Administration recently approved, is projected to save between $6 and $12 million by consolidating the liability insurance for the project.

Governmental Reorganization

How the government is structured, more specifically DOT, is integral to responding to the challenges of the 21st century. The advent of ITS and new financial options, as well as other more effective practices, require changes in how the DOT is organized. Such changes will enable the department to operate in a new environment, cost less and work better for transportation consumers.

In April 1995, Congress was sent a bill that will reorganize DOT. This proposed bill carries forward the commitment of the present administration to positive change.

Even though DOT has been a leader in initiating reform within its structure, a fundamental rethinking of how the department operates has been required. In the last few years DOT has:

- Cut back its civilian work force by more than seven percent, racking up savings of more than $260 million per year in personnel costs alone; and,

- Improved customer service through automation and by streamlining procedures and regulations.

However, efforts such as those still will not be enough to meet the challenges of the future. The problem lies with DOT's organizational structure. Currently, DOT comprises ten separate operating administrations. This antiquated structure hinders the department's ability to foster creative partnerships, to make strategic transportation investments and to fashion innovative mechanisms.

The proposed reorganization legislation would address these shortcomings by consolidating DOT's agencies into just three:

- A revamped Federal Aviation Administration;
- The Coast Guard; and,
- A new Intermodal Transportation Administration (ITA) that would integrate all of DOT's surface transportation and civilian maritime functions.

This proposal is intended to achieve three key results:

1. It repositions DOT to help develop the transportation system of the 21st century. It promotes *intermodalism*, which will increase efficiency and create a seamless transportation system. Adopting an intermodal approach is essential if the goal is to make the most of the existing transportation infrastructure in an era of limited new construction.

2. The reorganization will help DOT better serve its customers (from commuters to transportation professionals) by giving them what amounts to *one-stop shopping*. Currently, the department has multiple agencies with overlapping concerns. This fragmentation and duplication creates inconsistencies and a lack of coordination that frustrates users, other governmental agencies and affected parties. Integrating *all* of the surface transportation agencies into the new ITA will provide a way to end this frustration.

3. Reorganization will also help DOT to responsibly reduce the department's size, thus saving taxpayers money. It eliminates the duplication and incompatibility that comes from having ten separate agencies — each with its own personnel office, its own procurement department, and so on. Restructuring will help DOT meet its commitment to reduce its workforce by 12 percent. It will also permit a 50 percent cut in back-office administrative staff. The aim of these changes is to support and protect personnel who are serving department customers directly on the front lines.

It is estimated that the reorganization will save more than $1.5 billion over five years. The largest part of these savings will result from the proposed administrative changes, while improving service.

A Climate of Reform

Separate from the reorganization proposal, a statement of principles for the reform of transportation funding program was also submitted to Congress. While this statement is not legislation, it lays the foundation for subsequent legislative action.

Currently, DOT has more than 30 different programs for transportation infrastructure alone. Each of these programs has its own set of rigid rules, applications and review criteria. That so many different programs exist places an unacceptable burden on state and local governments as well as on private industry. This burden becomes more onerous in light of the fact that most of those entities must compete for, and make the most of, limited federal funds.

DOT's guiding principles for the reform of the current system are simple:

- Consolidation of the more than 30 infrastructure funding programs;
- Simplification of these programs' requirements; and,
- Increased flexibility and authority for governments at the state and local levels to determine which projects should receive federal funding.

The spearhead of this effort consists of beginning a dialog with Congress that will lead to the passage of funding program reform legislation. The reorganization legislation and the

DOT's statement of principles are what the department needs to meet the challenges of the 21st century. Otherwise, without the benefit of formulating a plan to meet those challenges, these efforts would seem to be without reason — in effect, budget-cutting and downsizing for the sake of budget-cutting and dismantling big government.

Summary

The aim of this restructuring plan is to make DOT more effective and more efficient. Therefore, it will permit the department, and its partners, to spend more time and energy keeping people moving, and less time and energy focused on moving paper — building bridges, not bureaucracy.

The strategies of ITS, innovative financing and government reform are at the heart of the Clinton Administration's ambitious plan to meet the challenges of transportation in the 21st century. The elements of these strategies — cutting bureaucracy and red tape; reforming programs; empowering states, localities and businesses; and increasing system efficiency — will permit DOT to continue its mandate to move people and goods effectively in an era of fiscal constraint.

NOTE — *This article was adapted from the author's Frank Keville memorial lecture given at a BSCES meeting on April 26, 1995.*

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The Ozark Mountain Highroad: A Highway Planning Model for the Future

A multi-agency partnership program based on the open sharing of information, coupled with an effective public participation program, are key to expediting project planning.

JERRY A. MUGG

In today's fast-paced world, nothing seems immune from being fast-tracked. The engineering community has developed many turn-key type services but they are targeted primarily only for privately funded projects. Transportation planning services have remained relatively unscathed by society's need to hurry up.

Customarily not known for its swiftness, the highway planning process, which must follow federal environmental review policies, has long been viewed as inefficient but necessary. Due to multi-agency coordination and public review requirements, highway planning projects have traditionally taken years to conduct. Upon completion, the highway agency then has authorization to begin the design phase, typically requiring an additional two to three years depending on the size and funding of the project.

The Ozark Mountain Highroad Project has changed that process. The Highroad, a planned 18-mile (29-kilometer), four-lane controlled-access highway on a new alignment, was conceived in June 1992. In January 1993, the Record of Decision was signed — after only eight months had elapsed. What many said could not be accomplished was completed — and in record time. By fast-tracking the highway planning process, the Highroad has shown that a highway project can be planned in an efficient and timely manner with complete regard to environmental considerations and public opinion requirements. The Highroad has become a model for the planning process for future transportation projects.

Project Setting

Branson, Missouri (population 7,700), is possibly the hottest entertainment center in the

country. Many nationally-known celebrities have established country music theaters along a four and one-half mile stretch (seven kilometers) of three-lane highway that winds along ridgetops in the Ozark Mountains of southwest Missouri. It is reported there are now more theater seats in Branson than there are on Broadway in New York City and on the famous strip in Las Vegas. In addition to the numerous entertainment attractions, the Branson area also offers recreational opportunities for both boating and fishing enthusiasts. With these natural and manmade amenities, the tourism industry in the area has experienced rapid growth over the past several years — overtaxing an infrastructure originally intended to serve a small rural community.

Because the region is rugged and mountainous, visitors to the area must rely on various forms of surface transportation and a limited highway system (see Figure 1). During the peak of the tourism season, 30,000 cars are jammed onto Country Music Boulevard (Route 76) each day. This congestion often results in average speeds of 10 mph (16 kph) for much of the day and intolerable delays. The current yearly delay is estimated to be over one million vehicle-hours and approximately 2.4 to 3.0 million person-hours. These delays translate into additional costs not only for the tourists, but also for the local and statewide economies.

Unique Solutions

In early June 1992, the governor of Missouri declared the traffic congestion in the Branson area an economic emergency that required immediate attention. The challenge presented by the governor to the Missouri Highway and Transportation Department (MHTD) was to plan a totally new transportation facility in six months without compromising safety (design parameters) or the integrity of the environmental review/approval process.

Planners for the Ozark Mountain Highroad devised a unique approach for accomplishing in a couple of years what normally requires five to seven years. The enormous need for a fast response to the transportation needs of the area was met with an unparalled partnering effort by agencies and consultants. This cooperation made it possible to compress the planning schedule to 30 percent of the time typically required for a project of this magnitude. Developed by the Federal Highway Administration (FHWA) and implemented by the MHTD and its planning consultant team, this unique approach to the Ozark Mountain Highroad highway planning process employed concurrent review procedures for the environmental documentation and other fast-track techniques not normally used in a conventional highway planning process.

However, time was not the only challenge. The difficult physical terrain of the project area and its physical restrictions required that planners be innovative in developing and evaluating alternatives. The difficult environmental terrain of the Branson area further constrained the planning effort since it necessitated that special studies be conducted to identify potential environmental impacts. Environmental issues surrounding the project were further heightened by increasing signs of stress created by the phenomenal growth of the area. Similarly, the unique nature of the project setting created an equally unique need for active public participation in the planning process. Consequently, a community involvement program was coupled with the Highroad planning effort to avoid confusion within the community concerning the fast-track approach and to augment the sharing of information.

Even with this multi-agency partnering approach, additional special techniques and practices had to be utilized by the Highroad planning team in order to bring concept to reality. These procedures complemented the partnering approach to maintain a commitment to the open process philosophy. Despite the special nature of the Ozark Mountain Highroad project itself, the tools, techniques and planning procedures employed can be applied to other projects that address transportation-related problems. The Highroad is an outstanding example of what can be done when a rapid response to a critical transportation issue is urgently needed.

Multi-Agency Partnering

The challenges of the governor's call to action — to fast-track the design process for the Ozark Mountain Highroad — were so great that a

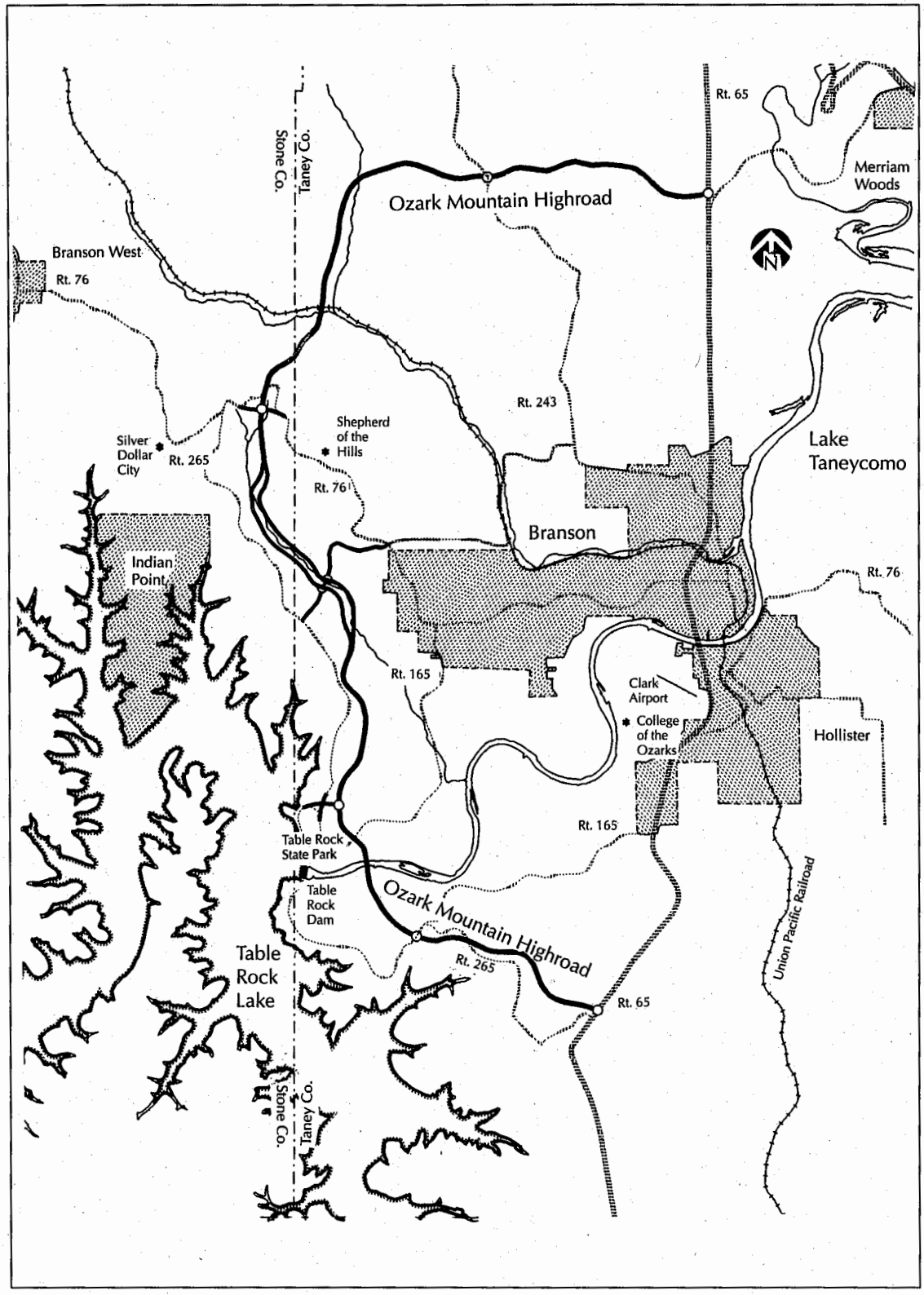

FIGURE 1. Branson, Missouri, is located in the heart of Ozark Mountain country.

typical, standard approach to the planning effort would have been inadequate. The demands and characteristics of the project setting required that a progressive approach to the planning and design process be taken. Realizing this, the Kansas City, Missouri, Division, and the Jefferson City, Missouri, District of the FHWA, in coordination with the Washington, D.C., office, developed a model program to advance the Highroad project in the most efficient and timely manner possible while maintaining compliance with the letter and intent of the National Environmental Policy Act (NEPA).

The cornerstone of this model approach was its open-process philosophy and call for multi-agency cooperation. The goal of the model program was to demonstrate that a highway project could be advanced in an efficient and timely manner while, at the same time, assuring that the best decisions were being made regarding not only the project itself but also the environment as a whole. To facilitate this goal, the open-process philosophy was developed to promote and encourage full consideration of all ideas and opinions, no matter how diverse, and to openly examine competing project needs.

After the philosophical cornerstone of the model program had been identified, efforts then shifted to how to establish and sustain the open-process methodology. To accomplish this, a partnering concept was developed and encouraged among the sponsoring agencies, the consultant team and the resource agencies. This partnership concept was based on pro-active scoping, open dialog and requests for active participation. A call for the active involvement of top level management also helped ensure a commitment to the program.

Shared ownership of the project by the multi-agency team was fostered by the acknowledgement of the individual and specific needs of each resource group. Through this process, a shared vision within the multi-agency partnership seemed to evolve, not only for the project itself but also to demonstrate that the whole process could move along efficiently. As the project progressed, the open-process philosophy seemed to feed on itself and keep the project planning process continually moving toward completion.

TABLE 1.
Planning Timeline

Days From Notice-to-Proceed	Product Delivered
18	First Public Involvment Meeting (Scope Meeting)
84	Preliminary Draft EIS Circulation
90-95	Public Open House Meetings
114	Draft EIS Circulation
135	Design/Location Public Hearing
190	Final EIS Circulation
234	Record of Decision

Procedures for a Model Environmental Project

To help guide these ideas into becoming a reality, a timetable for project development — termed "Procedures for a Model Environmental Project" — was developed. This timetable outlined the time constraints and lead agency responsibilities for the timely handling of tasks that would fulfill NEPA requirements. Time considerations for transportation and environmental enhancement components also were detailed. The timeline included the most crucial steps of the NEPA process from project inception to beyond the signing of the Record of Decision (ROD).

Key steps that required expeditious processing included:

- The notice of intent;
- The designation of agency coordinators;
- Requests for cooperating agency participation;
- Early notification to resource agencies concerning the project emphasis;
- The scoping meeting; and,
- The required steps of the environmental documentation preparation.

These steps included such activities as:

- Preliminary Draft Environmental Impact Statement (EIS) circulation;
- Draft EIS circulation;
- A public hearing;
- Final EIS circulation; and,
- The signing of the ROD.

The timetable for the environmental documentation preparation, including the corresponding notices of availability and public announcements, was established based on the minimum review time requirements prescribed by NEPA (see Table 1).

Therefore, little slack time was available for responding to official agency EIS comments. These conditions called for extraordinary cooperation from the multi-agency team. The model procedures called for a two-week turnaround for the review of the preliminary Draft EIS — typically not a prescribed step of the NEPA process. Similarly, requests for the expeditious review of the Draft EIS and the Final EIS, in the spirit of the open-process philosophy, were also submitted.

Consistent with the partnering concept upon which the fast-track procedures were founded, the planning consultant's responsibilities for the project were expanded to include establishing liaisons within the multi-agency team members. Information that was necessary for key decisions and other prescribed data circulation items were distributed by the planning consultant for the various agencies to review and respond simultaneously. This arrangement significantly improved the time needed to make collective decisions by the FHWA and the MHTD.

Breaking the Traditional Mold

Typically, highway planners strive to efficiently and safely transport users from one point to another. Roadway grades, alignments and configurations are designed to maximize system efficiency for the benefit of the user. The overlying principle is to provide a means of safe transportation with a design that is as cost-effective as possible. Physical, natural and environmental constraints sometimes dictate that there be some adjustment in the planners' logic, but for the most part a lack of regard is given to the highway's relationship with the natural surroundings. These factors are typically avoided — not due to an unwillingness on the part of the highway planner, but rather due to not being an objective at the outset of the planning effort. For the Highroad, this was not the case. At the outset, the Highroad project team identified the need to break the traditional mold.

Thinking of the typical motorist in the Branson area (a tourist drawn by the natural beauty of the Ozarks), an identity for the Highroad was envisioned at the outset of the project. The Highroad would be the gateway to the Branson area and would likely be the primary means for many to experience the Ozarks. The multi-disciplinary planning team decided to incorporate the Highroad into the physical terrain and to create opportunities for the public to experience the distinct Ozark surroundings in the Branson area. These opportunities included providing scenic overlooks, bicycle and hiking trails, and historical interpretive centers.

The planning of the roadway was unique in its attempt to blend the roadway grades with the physical features of the Ozark Mountain countryside. A parkway-type facility with a median that continually varied in width from seven to over 700 feet (2.1 to 213 meters) was developed. Split alignment grades were proposed for the opposing travel lanes so that opposing traffic cannot be seen as the opposing directions weave in and out of the rugged terrain. With the cooperation of the MHTD, these "relaxed" design standards (in comparison to a normal four-lane, access-controlled state highway) were used in order to achieve a very different type of highway (see Figure 2).

Billboard control provided another means of integrating the Highroad into the natural Ozark terrain. The Missouri Highway and Transportation Commission designated the Highroad as a "Scenic Byway," which, as part of the primary state highway system, allows for the regulation of all outdoor advertising that is visible from the highway right-of-way. This designation permits state control over the proliferation of outdoor signs along the roadway that might interfere with Highroad users' enjoyment of the natural beauty of the surroundings. This control is being accomplished with the purchase of scenic easements that run par-

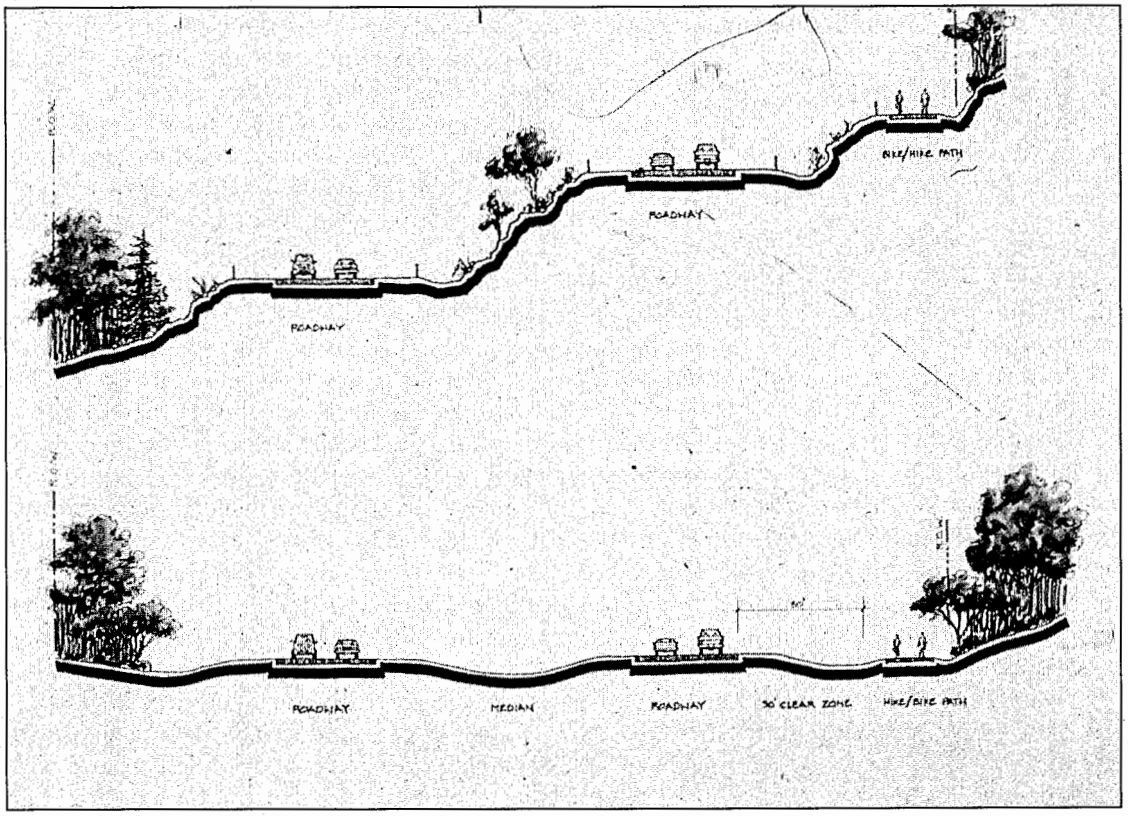

FIGURE 2. The Highroad blends into the rugged landscape using split horizontal and vertical alignments.

allel to, and 660 feet outside, the Highroad's right-of-way line.

Up-Front Commitment to Enhancement

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 put into action a national policy that sought to blend transportation facilities into communities and the natural environment. Transportation programs today must be compatible with national environmental goals. ISTEA has strengthened the environmental aspects that must be considered during the decision-making process and encourages the preservation and/or enhancement of the affected environment. These enhancements take many forms. From guidelines to help preserve wetlands preservation to scenic byways and pull-offs, ISTEA recommends the incorporation of many features to support the environment into the design of new transportation facilities.

In the case of the Ozark Mountain Highroad, a commitment to a complete and full investigation of the enhancement opportunities available through ISTEA was established at the outset of the planning effort. This commitment, and its implementation, reassured the various resource agencies of the planning team's intention to provide the best possible transportation facility in response to the needs of the Branson area and that this facility would take full advantage of the features that can enhance and promote the natural environment.

At the outset of the project, the planning team recognized that the cultural significance of the area represented an enormous opportunity. The Branson hills are rich with historical treasures waiting to be uncovered. With a traditional planning schedule, the typical approach would have been to investigate specific route alignments for items of cultural and historic significance. These sites, if deemed significant, would most likely have been avoided by

the proposed action with only minimal (if any) expenditure for mitigation.

In the case of the Highroad, time would not permit this approach. As soon as the project began, archeological teams were scouring the ridgetops and hillsides within a 4,000-foot (1.2-kilometer) wide corridor where the Highroad was most likely to be located in search of historical or archeological remains. An agreement with the State Historic Preservation Officer was signed that committed the planning team to conduct an investigation of the entire corridor in accordance with prescribed procedures and documentation requirements.

As the project progressed, significant sites and findings were shared with the planning team so that adjustments could be made to the alternative routes under consideration. Meetings were held with the appropriate agencies to discuss subsequent data recovery, mitigation and possible preservation efforts. A willingness to pursue the possibility of cultural interpretation facilities in conjunction with the Highroad was articulated and specified in the Final EIS. With this up-front cultural commitment, potential delays in the project were avoided and the objectives of the Highroad were able to merge with the goals of the historic preservation program.

Other commitments made during the beginning of the planning process also helped move the project forward. During the wetlands coordination phase conducted in concert with the U.S. Army Corps Corps of Engineers, a willingness to consider employing methods to enhance and preserve wetlands was expressed even if such efforts were not required. Indeed, a commitment was made in the Final EIS to pursue the creation of special aquatic sites (wetlands) within the Highroad right-of-way as part of final design even though these efforts were not required to mitigate any expected negative impacts.

Similarly, a commitment to the preservation and enhancement of wildlife habitats throughout the right-of-way was made. A comprehensive landscaping and wildlife habitat master plan was drawn up. The extra-wide median areas designed for the Highroad not only provided a unique visual aspect to the Highroad, but also provided planners with the opportunity to integrate wildlife habitat management features into the highway's construction. A commitment to work with the resource agencies in further pursuit of these goals was made in the Final EIS. The multi-agency partnering concept will go on through the final design process.

Progressive Resource Scoping

One of the elements of the NEPA compliance process is the sharing and gathering of information to identify problems and constraints associated with the proposed action. This process, called *scoping*, has long been a normal component of the EIS preparation process.

The goal of scoping is to provide the opportunity for public and private agencies to share their views on various social and environmental issues that are considered to be affected by the proposed action. These ideas can then be evaluated by the transportation planner to shape the proposed action. Scoping should be initiated in the early stages of the planning effort so that all critical issues are known as soon as possible, thereby allowing the planner ample time to evaluate all the issues. Early scoping also helps ensure that time is not wasted on pursuing issues of low significance.

The NEPA regulations encourage innovation with regard to project scoping since the specific requirements of the regulations that cover scoping are limited. However, the regulations do specify that an open scoping process with public notice should be provided. The scoping process should:

- Include the identification of both significant and insignificant issues;
- Identify the related analysis requirements; and,
- Present a schedule for the EIS preparation.

On June 26, 1992, at the MHTD offices in Springfield, Missouri, a formal scoping meeting for the Ozark Mountain Highroad was held. This meeting was announced publicly and was attended by federal, state and local agencies and conservation groups. Information packets with personal invitations to the resource agencies had been mailed prior to the

scoping meeting. A brief overview of the project and descriptions of the alternatives being considered were provided. Other items discussed included the "Procedures for a Model Environmental Project" and the project's environmental considerations. All of these discussions were preceded by an invitation by the MHTD District Engineer, during his opening remarks, for cooperation and openness among all of the attending agencies.

However, the scoping meeting did not mark the date when the scoping process for the Highroad actually began. Prior to the formal meeting, hundreds of hours had already been spent in the field or on the phone talking and meeting with the various resource agencies in an effort to gather information and identify needs. When the official notice-to-proceed was received on June 8, 1992, the consultant team — consisting of transportation planners, foresters, geologists, environmental scientists, archaeologists and engineers — hit the ground running. Time could not be wasted waiting for the environmental scoping issues to arise at the formal scoping meeting. A progressive approach to the scoping process was needed.

For the multi-agency partnering relationship to work, the scoping process for the Highroad project had to promote and complement the open-process philosophy. Individual meetings with the resource agencies were held to encourage the open sharing of information.

Field reconnaissance meetings with the U.S. Army Corps of Engineers and the Missouri Department of Conservation (MDC) were held to gather relevant data. The Corps, a cooperating agency for the Highroad EIS process, designated a project contact person within the agency specifically for the Highroad project. Several meetings with the Corps were scheduled in the field to delineate the location of, and assess how the highway would affect, the wetlands within the project area. As is the responsibility of the Corps in the administration of the Clean Water Act, Section 404, it provided guidance on the regulatory requirements and their implications on the proposed action. This input was received early enough in the process so that the planners had ample opportunity to make adjustment for these conditions.

The tight environmental constraints that were imposed on the project corridor in the vicinity of the Lake Taneycomo crossing required early scoping with the MDC. The MDC, in a lease agreement with the Corps, owns and operates the Shepherd of the Hills Fish Hatchery located along the north bank of Lake Taneycomo immediately downstream of the Table Rock Lake Dam. Since it was located within the Highroad study corridor, planners determined that avoiding the Hatchery site (a Section 4(f) resource) was not feasible or prudent (see Figure 3).

Consequently, members of the consultant team met with the Hatchery manager and naturalist to ascertain what impacts on the Hatchery site would be acceptable. Areas of unacceptable consequences were pointed out so that some of the alternative alignments could be eliminated from further study.

The MDC scoping also provided valuable input as to what ancillary features the Highroad should offer as it passed through the Hatchery area. For example, a closed drainage system with spill containment ponds for the Lake Taneycomo bridge decks was planned in response to MDC concerns.

Also, input was received on the desired style and aesthetic features of the lake bridge. Using this progressive approach to scoping, planners were able to adequately address the Hatchery concerns in the EIS documentation and Section 4(f) Statement so that the reviewers of the document were satisfied.

The planning team met with many other individuals or groups including:

- Staff from the Ruth and Paul Henning State Forest;
- The NEPA coordinator and wildlife specialist from the U.S. Forest Service;
- Table Rock State Park staff;
- U.S. Fish and Wildlife Service staff; and,
- Taney County and Stone County planners.

These meetings provided invaluable data concerning such issues as impacts on wildlife, habitat fragmentation, threatened and endangered species, and land use concerns. As a result, before the scoping meeting was even held,

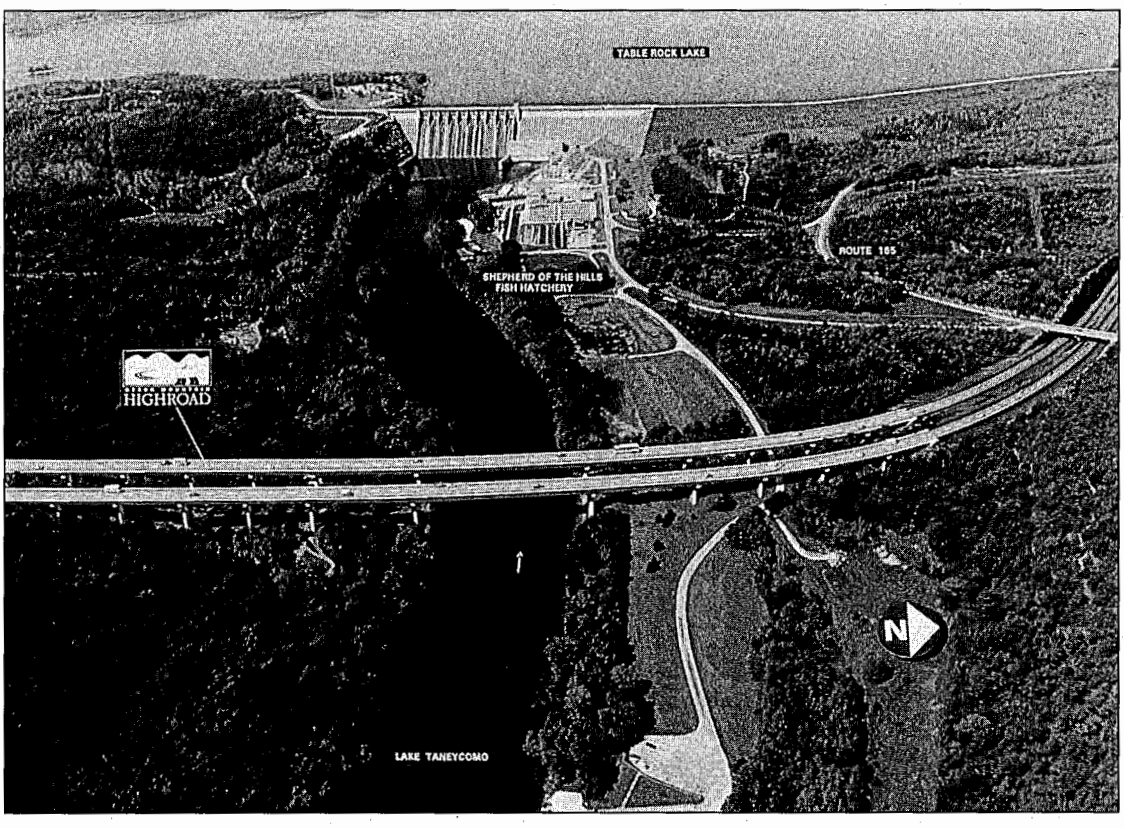

FIGURE 3. Through progressive scoping, an acceptable alignment, with appropriate mitigation measures, was identified for the Highroad near the Shepherd of the Hills Fish Hatchery.

an action plan to resolve the identified environmental concerns was in place and moving forward.

In response to concerns about impacts to the water quality of Marvel Cave (a designated critical habitat for the gray bat, which is on the federal government's endangered species list), an expert on hydrogeology was brought on board to determine if the Highroad corridor lay within the Marvel Cave recharge area. Similarly, a biologist with expertise in cave life was hired to assist the team by investigating the caves that were discovered within the corridor by the geologic reconnaissance team. These actions allowed the planning team to show at the formal scoping meeting how these issues and problems were already being tackled.

Community Involvement Program

One of the key elements of the Highroad project that greatly facilitated and complemented the multi-agency partnering approach was its active community involvement program. At the outset of the project, the need for a progressive approach to public involvement was recognized. The fast-track schedule for the project and the difficult public relations setting of the Branson area created an enormous challenge for the community involvement program. Since the project was receiving high-profile status in the local media, a program was needed to reassure the public that their needs and concerns were being addressed. Similarly, with the governor's charge to plan the highway as quickly as possible, the public needed assurances that important steps were not being omitted. The community involvement program was designed to address these issues.

There were essentially six elements to the Highroad community involvement program:

- Project identity;
- Public opinion research;
- Media relations;

- Public affairs;
- Public information; and,
- Location/design public hearing.

In order to develop and structure a communications plan for the Highroad project, researchers worked to determine the range of questions and concerns held by the people affected by the Highroad. Their research was conducted utilizing three focus groups that were composed of local residents and small business owners in the Branson area. The goals of the communications program included sharing information with the public about the ongoing studies and investigations being performed for the Highroad as well as educating the public about the planning process.

The Highroad community involvement program also consisted of an active public affairs effort. Routine, periodic news releases about the Highroad studies were provided to local news publications. A full-time community involvement coordinator was hired to provide on-site public information services from the outset of the project until the public hearing process had finished. Other activities included establishing a telephone message center at the project office to handle the public's questions about the project.

A public information campaign helped disseminate project information as it became available. A fact book about the project, prepared in a question-and-answer format, was distributed to the public. Numerous "advertorials" addressing issues related to the project were published in local newspapers. In addition, a multi-color, six-fold brochure wrapped up the public information program for the planning phase.

One of the aspects of the public information program created specifically for the Highroad was a series of open house meetings conducted by the MHTD and the consultant team. These meetings were held after the publication of the preliminary Draft EIS and before the formal Draft EIS. Since the fast-track schedule required simultaneous planning and design efforts, public input was needed at an early stage so that the design of the alternative most likely to be selected as the preferred alternative could be initiated while the planning efforts continued. These open houses provided the team with this bridge of public information prior to the official public hearing so that work on preliminary designs could commence. As a result of the open house meetings, critical decisions about the location of the Highroad were made by the planning team.

The final element of the community involvement program involved the public hearing for the Highroad. Because the preliminary design of the preferred alignment was being developed simultaneously with the planning effort, the public hearing covered two issues — corridor location and right-of-way design.

Also, in lieu of the traditional format of the standard public hearing process typically employed by the MHTD, it was decided to use a new public hearing format for the Highroad. Given the tremendous wealth of information the team had compiled, it was decided that an open-house format for the public hearing would be most appropriate. The open-house format promotes public participation and eliminates the traditional presentation and subsequent question-and-answer session.

On October 21, 1992 (135 days following the consultant's notice-to-proceed), the combined location and design public hearing for the Highroad was held. Based on the public's responses, the open-house format was well-received and the project benefitted by the open sharing of information. A total of 327 comments were received during the public hearing process.

Proactive Team Management

Managing any project with the intricacies and multi-discipline coordination requirements of the Highroad can be demanding. Add fast-track scheduling and the demands become extraordinary. With fast-track scheduling, little time is available for those needed adjustments in the management plan that always arise. Under these circumstances, keeping focused on the overall objective and maintaining work toward that objective are crucial. Such was the case with the Highroad.

One of the key ingredients to the success of the Highroad planning effort was the forward thinking of the management team. Using the "Procedures for a Model Environmental Pro-

ject" as a road map for the timing of key events, team management identified key intermediate milestones for decisions or for beginning certain tasks. One example of this foresight was the series of team decisions leading up to the ultimate selection of the preferred roadway alignment alternative. Initial screenings, preliminary screenings and final evaluations were timed to provide the decisions needed to move the project forward. For each decision, necessary information concerning environmental effects was provided so that the decision-makers were in a position to make wise and careful judgements.

This decision-positioning management style was also employed in the project progress meetings. Bi-weekly project progress meetings were scheduled throughout the duration of the project. An overall schedule of these meetings was developed so that the goals and objectives of each meeting were mapped in advance. This method allowed the team adequate time to prepare for each meeting's agenda and to ensure that needed decisions were being properly handled.

Other key factors contributing to the planning effort's success included the identification of a sound and well conceived purpose and need statement for the proposed action. A strategy for the process of selecting the preferred alternative from the multitude of alternatives was developed based on the statement of purpose and need. The fulfillment of this statement then became the litmus paper for the selection or rejection of an alternative. The statement of purpose and need became the foundation for the environmental documentation upon which the study was built.

Common project management practices and tools — such as task arrow diagramming, manpower projections, cost controls, and progress reports — were also utilized. Progress reports and action reports that contained graphical presentations charting actual progress versus planned progress were prepared on a bi-weekly basis. The action report summarized activities of the planning team in the past weeks and outlined activities to be completed in the following weeks. Both of these reports were helpful management tools throughout the duration of the project.

A Highway Planning Model for the Future

Every transportation project is unique in one way or another. Project surroundings and circumstances that precipitate the action combine to create the unique aspects of any project. Unique problems call for unique innovative solutions. This maxim is especially true in the case of the Highroad. Urgent needs and heightened public awareness called for fast action and the Highroad planners were up to the challenge.

But can the special techniques and approach employed by the Highroad planners be used again? Can planning processes conducted under the context of NEPA be expedited on other projects? The answer is yes. Under the proper circumstances and setting, what the Highroad accomplished can be duplicated. Whether as a whole or in distinct pieces, the planning approach of the Highroad can be applied to new problems and new challenges. The Highroad has become a model for other planners to follow.

For the Highroad project, the binding element that created and sustained the multi-agency partnering and open dialog was the urgent need for action. Public announcements gave credence to the process and enforced the shared vision of the participants. Priorities shifted in upper management and unparalled commitments were made to the project. The urgency of the action affected all aspects of the planning process and gave purpose to the fast-track procedures.

Regardless of the degree of urgency, the "Procedures for a Model Environmental Project" and partnering concept can be used on other projects. The degree of fast-track success may vary depending on the urgency of the specific proposed action and the details of the timeline should be tailored to the circumstances surrounding the project.

All problems are different and what was accomplished on the Highroad may be too much to expect for every project. But the Highroad has shown that by utilizing a partnering approach, a highway project can be planned in an efficient and timely manner with complete regard to environmental considerations and

opinions. The traditional planning approach can be expedited. The planning and preliminary design efforts for a highway project can be successfully joined into a process that can speed up the traditional progression of concept to construction.

Utilizing management tools such as non-traditional design concepts, progressive resource scoping, a community involvement program, proactive management techniques and up-front enhancement commitments, other projects can also accomplish in two years what used to take five to seven years.

Finally, the procedures and openness created by the Highroad planning effort must go on. Environmental obligations do not end with the signing of the ROD. As the Highroad now moves into the final design phase, the open dialog and coordination with the various resource groups, both public and private, continues. The unique approach successfully developed for the planning of the Highroad must not be abandoned as the Highroad moves on a fast-track pace to reality.

ACKNOWLEDGMENT — *HNTB Corp. headed MHTD's consultant team that developed and implemented the fast-track planning process.*

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It's a Pratt! It's a Howe! It's a Long! No, It's a Whipple Truss!

Over the last century the nomenclature for certain bridge design elements has been based on error, not fact. It is time that the record be set straight.

FRANCIS E. GRIGGS, JR.

In the 19th century new truss patterns were usually named after the man who received the patent or was its primary builder and proponent. By the 20th century some of these trusses were called by names that do not necessarily accurately reflect their genesis. While engineers have become accustomed to using these names, a good deal of the history involved in the development of these trusses has fallen by the wayside. Many great engineering minds of the 19th century have been been lost to our engineering vocabulary in this manner, almost creating a form of revisionist history. The exact knowledge of the contributions of the original creators of truss patterns — specifically Stephen H. Long, Squire Whipple, Thomas and Caleb Pratt, and William Howe —should remain with us.

Stephen Harriman Long

Stephen H. Long, of the United States Army, was granted his first bridge patent in 1830 and his last in 1858 (see Figure 1). Twenty-four patents dealing with bridges had been granted by the patent office prior to Long's first patent (see Table 1). Not all of these patents, however, dealt with truss bridges. Unfortunately, as a result of a fire at the patent office in 1836, the nature of some of the patents is unknown. Palmer, Burr, Town and Wernwag were the leading wooden bridge builders of the early 19th century. Finley and Templeman were known for their chain bridges and Pope for his "Flying Pendant" cantilever bridge.

Unlike most of his contemporaries in the bridge building business, Long was a college graduate and well versed in mathematics. He graduated from Dartmouth College in 1809 at the age of 24 and taught mathematics at West Point for several years prior to being assigned to the Topographical Engineers. It was during his assignment to the Baltimore and Ohio Railroad from 1827 to 1830 that he became interested in the design and construction of bridges. In his 1829 monograph "Rail Road Manual or a Brief Exposition of Principles Applicable in Tracing the Route of a Rail Road" that was reproduced in part in the *Journal of the Franklin*

FIGURE 1. Stephen Harriman Long.

Institute, he wrote that "the mode of construction deemed most applicable, is that adopted by Mr. Wernwag, so far as [it] relates to the formation and adjustment of the main arches." At the time of publication he had evidently not given much thought to the topic of bridges. After leaving the railroad, however, he received patents on six bridges (see Table 2).

Long's Jackson Bridge

Long's first patent was for a 109-foot span roadway bridge he built on the Washington road, about two and a quarter miles from Baltimore, where it crossed over the Baltimore & Ohio Railroad. This bridge, named the Jackson Bridge, was a parallel chord, rectangular profile truss with diagonals in compression and verticals in tension (see Figure 2).

To give the truss additional support, he installed two braces (he called them "inferior arch braces") off of the abutments out to the first and second panel points. In addition, he

TABLE 1
Early Bridge Patents

Patent Holder	City	State	Date	Patent Number
W. Peale	Philadelphia	PA	January 2, 1797	NA
C. Fowler	Philadelphia	PA	February 24, 1797	NA
J. Stickney	Worcester	MA	June 3, 1797	NA
T. Palmer	Newburyport	MA	December 17, 1797	NA
T. Burr	Oxford	NY	February 14, 1806	NA
T. Pope	New York	NY	April 18, 1807	NA
J. Finley		PA	June 17, 1808	NA
J. Templeman		MD	August 16, 1808	NA
J. Templeman		MD	March 6, 1810	NA
J. Dyster	Philadelphia	PA	February 23, 1811	NA
J. Jessup	York(town)	PA	July 1, 1811	NA
R. Crosbie	Newark	NJ	February 17, 1812	NA
L. Wernwag	Phoenixville	PA	March 28, 1812	NA
B. Connor	Portsmouth	NH	April 23, 1812	NA
G. Tabb	Martinsburg	VA	February 23, 1816	NA
T. Burr	Burr Haven	PA	April 3, 1817	NA
N. Bishop	Barre	VT	January 11, 1819	NA
J. Bragg		Canada	January 4, 1820	NA
I. Town	Fayetteville	NC	January 20, 1820	NA
M. Lewis	Chenango	NY	April 12, 1820	NA
W. Woomansee		VT	March 6, 1827	NA
G. Wilkinson	White Creek	NY	May 15, 1827	NA
T. Blakewell	Pittsburgh	PA	May 15, 1827	NA
L. Wernwag	Jefferson	VA	December 22, 1829	NA

added "superior arch braces" or "those applied at the top of the truss frames, centrally of the span, and serving to relieve the upper strings of a portion of the thrust to which they are subjected."[1] In his 24-page pamphlet (that included diagrams) published in 1830 entitled "Description of the Jackson Bridge Together With Directions to Builders of Wooden or Framed Bridges," he included two articles that had been published in the *Journal of the Franklin Institute* as well as testimonials and "Directions to the Builders of the Jackson Bridge." In this publication, one of the first to specify actual member sizes for a truss bridge based on a crude knowledge of loads in the members, he presented in two tables the member dimensions for spans of 60 to 300 feet (along with a column of data entitled "load that may be sustained on the bridge"). His string and arch pieces were uniform in size in all panels and consisted of three members, of which the center of the three pieces was approximately 50 percent larger than the two side pieces.

He wrote that "the spaces between the posts (panel lengths) are in all cases equal to two-thirds the height of the bridge between the strings, which is believed to be the best proportion, to be preserved in arranging the relative distances between the upper and lower strings, and the posts."[1] Squire Whipple, to be discussed later, found this ratio to be one to one, or diagonals on a 45-degree angle. Long wrote that "stiffness can only be insured in a braced trestle (truss) by braces and counterbraces"[2] — something else that Whipple was to challenge later. Long considered the posts, main braces, counter braces in his detailed analysis as follows:

TABLE 2
Long's Bridge Patents

Patent Number	Date
NA	March 6, 1830
NA	January 23, 1836
1,397	November 7, 1839
1,398	November 7, 1839
5,366	November 13, 1846
21,203	August 17, 1858

Posts: "The size of the posts for a bridge less than 120 foot span need not exceed 6 inches square for the *quarter* posts, and 6 by 8 inches for those at the center and extremities of the bridge." It is clear from this requirement that he did not have a clear understanding of the load in the verticals that increases under full and most partial loading from the center towards the end of the truss.

Main Braces: "Their dimensions transversely, should be the same as those of the *quarter* posts." The same lack of understanding of forces in the web members is clear.

Counter Braces: "These should be equal in size to the main braces . . ."

Stay Braces: "The stay braces are essential not only to an erect posture, but to the longitudinal stiffness of the bridge; and ought to be introduced not only at each abutment, but

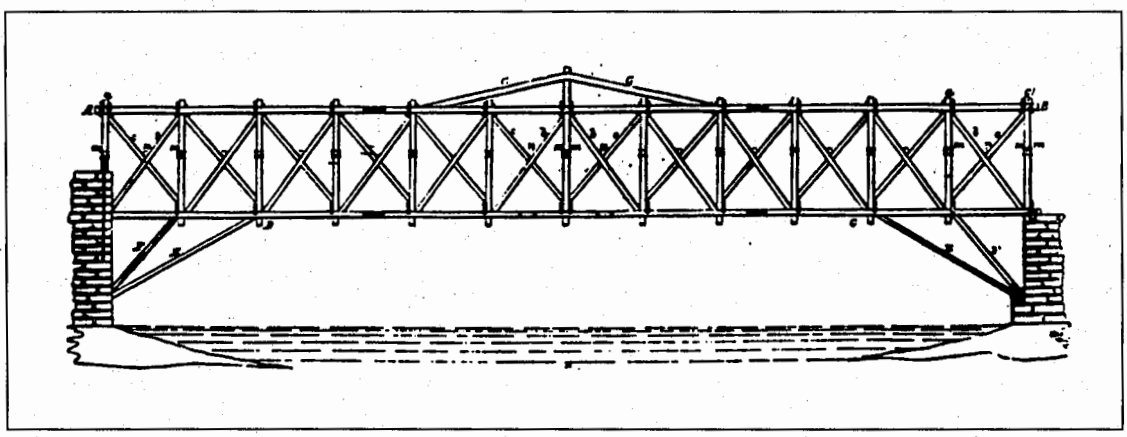

FIGURE 2. Long's 1830 Jackson Bridge.

at every pier of the bridge. They may be made to perform their office either by tension or thrust. In the former case, they may consist of iron rods, extending from the abutment or pier beams, through the posts at the extremity of the span; and to an anchor, or other fastening, in the abutment or pier. If the latter they may be connected to the end or pier posts, at the lower extremity of the transverse braces, or at any other convenient point, and extend to a sill or other suitable fixture in the abutment or pier. . . . The size of the iron stay brace, need not, in ordinary cases, exceed one and a quarter inches square. . ."[1]

It should be noted that the stay braces are mounted perpendicular to the line of trusses and serve only the function of keeping the trusses vertical, in contrast to stays which in suspension bridges help to carry the vertical loading. Long further claimed that:

"[M]oreover, the principles aimed at in the construction of this bridge, are such, that the strain to which the truss frames are subjected by the heaviest load, that is admissible upon the bridge, is no greater than that exerted upon it, without any load at all. Paradoxical as this may appear, it is, nevertheless, demonstrably the fact, with respect to all parts of the bridge, except the arch braces, and those parts merely, which are in contact with the sleepers or bolsters upon which the bridge is sustained."[1]

He clearly expressed the structural behavior of his truss by writing:

"The system embraced in the Jackson Bridge is such, that the braces all act uniformly in the direction of their axes, and exclusively by *thrust*. Their connexion with the truss frames is such as to preclude any action by *tension*. . . . A structure possessing these properties cannot fail to recommend itself to the consideration of those who may be interested in the adoption of the most firm and substantial mode of constructing bridges. Nor is the simplicity and economy displayed in the construction of the Jackson Bridge less conspicuous than its firmness and efficiency."[2]

His consideration of span length was also ahead of his time. Using the example of a stream or river crossing (instead of a railway crossing as in his first bridge), Long wrote:

"The extent of the spans of bridges should be regulated by the size of the stream, and especially by the quantity of drift and ice likely to be brought down in times of freshets. In dividing the breadth of a river into spans for a bridge, it will be well to make them as nearly equal as possible, except the exterior spans, or those contiguous to the contemplated abutments, which should have only about three-fourths the extent of the other spans. The reason for this will be obvious from a recurrence to the principle of double action in the strings — as also from the circumstance that the trouble and expense of trussing upon the abutments for the purpose of giving tension to the upper strings may thereby be saved. In conformity to the arrangement here adverted to, the center posts of the truss frames for the exterior span must be located not at the centre of the spans, but at a point distant from the abutment about one-third part of the length of the exterior span."[1]

It is clear that Long was considering making his trusses continuous over the interior spans, thus making the top and bottom chords both tension and compression members depending on the location of the live loads. He also recommended that "the splices should be as remote as possible from the points of greatest tension, *viz:* from a point midway of the lower string, and from points immediately above the piers in the upper strings."[1] In addition, he claimed that:

"However slender the materials of which this bridge is composed and however deficient in strength it may appear, it is crossed daily by stages at full speed, and has actually sustained about eighty beeves, driven across it at once, in close gang, without the least apparent yielding in the truss frames. Agreeably to the most approved rules for comput-

ing the strength of similar structures, it will sustain, on every square foot of its floor, in addition to its own weight, at least 120 pounds, or equally distributed over the entire surface of the floor, about one hundred and ten tons weight."[1]

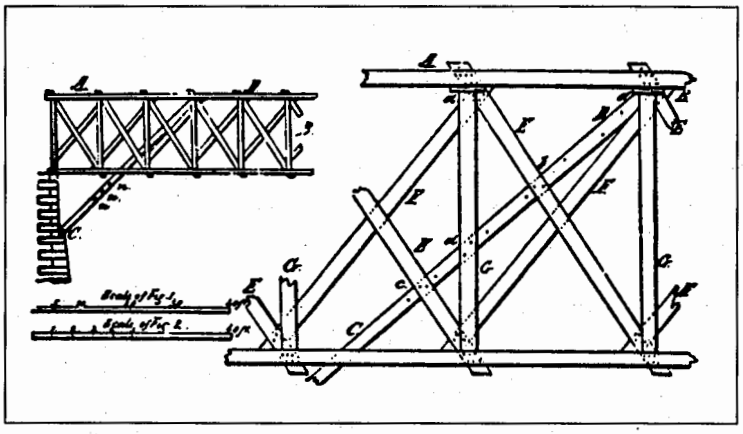

FIGURE 3. Long's 1839 suspension bridge.

To illustrate his design he built a 20-foot-long model of his bridge and presented it to the Franklin Institute at its meeting on June 24, 1830. The pamphlet he published that year was a remarkable publication since it was the first (or at least it was an early) description of a truss that had been designed by a practicing engineer. That pamphlet, along with his two Franklin Institute articles, placed Long in the forefront of early 19th-century bridge engineers. No document was published of such significance until 1842 when Herman Haupt published his "Hints on Bridge Construction."

Long's 1839 Patent

The most significant of Long's patents was his first 1839 patent for a wooden-framed suspension bridge (see Figure 3). In his patent application, and subsequent pamphlet, he stated:

"The suspension bridge is composed of two or more truss frames, together with arch braces, lateral braces, flooring, *etc.*, and is distinguished from other bridges heretofore invented and now in use by reason of the following actions on the posts, main braces and counter braces of this truss frame: to wit, *the posts act by thrust instead of tension, and the main and counter braces by tension instead of thrust,* [emphasis added] as in other bridges. Of course, the relative position occupied by the main and counter braces in the suspension bridge are completely the reverse of those occupied by them in common bridges; and the modes of attachment between the several parts of the truss frame, are materially different from those of other truss frames. . . ."[3,4]

In the same application he indicated his knowledge of structural behavior by demonstrating how his truss could have its panel spacing varied in such a way that the diagonals were under equal tension and, therefore, could be of similar dimensions. He considered a truss with a span of 120 feet and panel lengths varying from 18 feet at mid-span to 5.25 feet at the abutments. If the two double panel counter braces were removed, with the two central panel counter braces left in, the drawing would appear as shown in Figure 4.

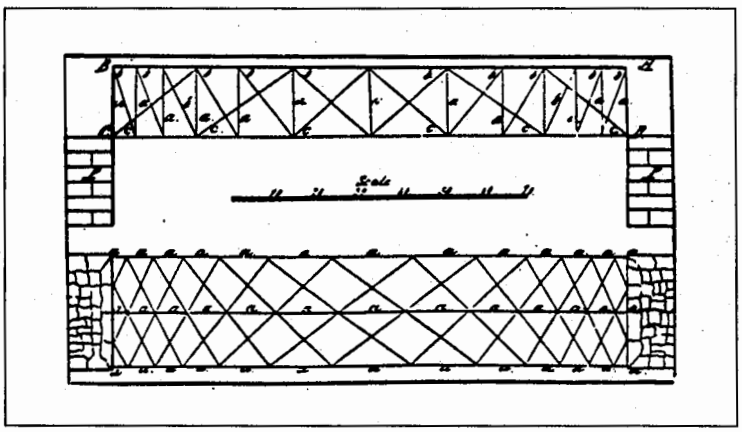

FIGURE 4. Long's truss bridge with equal loading in the diagonals.

He also wrote that his invention was:

"[N]ew and original; not only with respect to construction of wooden bridges, but also with respect to bridges composed of iron, or partly of iron and partly of wood, which may be constructed of similar parts nominally, though these parts may all differ in shape, dimensions, and manner of attachment to each other, all of which may be varied according to circumstances."[3]

In a pamphlet written in 1836, Long expanded on his Jackson Bridge and discussed his new lateral bracing system and a series of movable bridges he had designed.[4] He also gave his procedure for determining the length of spans for a long bridge. He recognized the importance of continuous trusses, introducing the terms *single action* and *double action*. Single action refers to simple span bridges and double action to bridges in which the chords were continuous over the interior supports. His analysis indicated that a bridge could carry twice the load at mid-span if it was continuous rather than simply supported.

In another pamphlet he published in 1839, and which was reprinted in the *Journal of the Franklin Institute*, he finished with the following statement regarding the use of his suspension bridge in iron:

"It is obvious that the arrangements prescribed in reference to the suspension bridge, are not only applicable in wooden structures, but, with slight alterations and appropriated modifications, the same principles are equally applicable in the construction of iron bridges. In this application of the principles, it would be advisable to construct all the parts that are exposed to *tension*, of wrought iron, while those exposed to *thrust*, or compression, may be constructed either of wood or of cast iron. The connexion between the strings, main and counter braces, may be effected by means of key-bolts of suitable size, passing entirely through these parts, and confining them together, the main and counter braces being merely rods or bars of wrought iron with eyes at their extremities, adapted to the reception of the bolts."[5]

In 1839, Long, therefore, had patented a bridge in wood (which would also work in iron), in which the diagonals were in tension and the verticals in compression. He could adjust the camber in the bridge by means of wedges at the ends of its vertical posts. Another patent, issued on the same day, was for a bridge with the same profile but with compression braces and tension/compression counterbraces and verticals in tension. It is clear that he also considered a truss with both diagonals in compression and verticals in tension, in his Jackson Bridge as well as in this 1839 patent, thus predating William Howe's patent by eight months.

That Long knew how loads were resisted by truss diagonals was indicated in an article he published in the *Journal of the Franklin Institute:*

"2nd. The mode of bracing the trestle or frame pier is often defective and inefficient. The braces are frequently arranged in such a manner that the thrust or resistance communicated by them is met by no other counteraction than that imparted by the stiffness or inflexibility of the beam or post with which they are connected. This must be regarded as a defect totally incompatible with firmness of structure, and should be studiously avoided. The remedy is simple and obvious, and is at once suggested by a recurrence to the principle denominated the 'parallelogram of forces.' The action of the brace, whether by thrust or tension, should be communicated to the sides and ends of the parallelogram in such a manner as to resolve itself into, or be counteracted by two forces acting at right angles to each other, and at the same point. A wooden brace should generally act by thrust, consequently the post, and beam, or stile, that receives its action should act by tension. From the principles here adverted to, it may be inferred that stiffness can only be insured in a braced trestle by braces and counterbraces, the heads and feet of which are firmly connected by ties, also that a trestle cannot be rendered firm and unyielding when the braces meet the post, beam or sill, at points unconnected by ties in the manner just explained."[2]

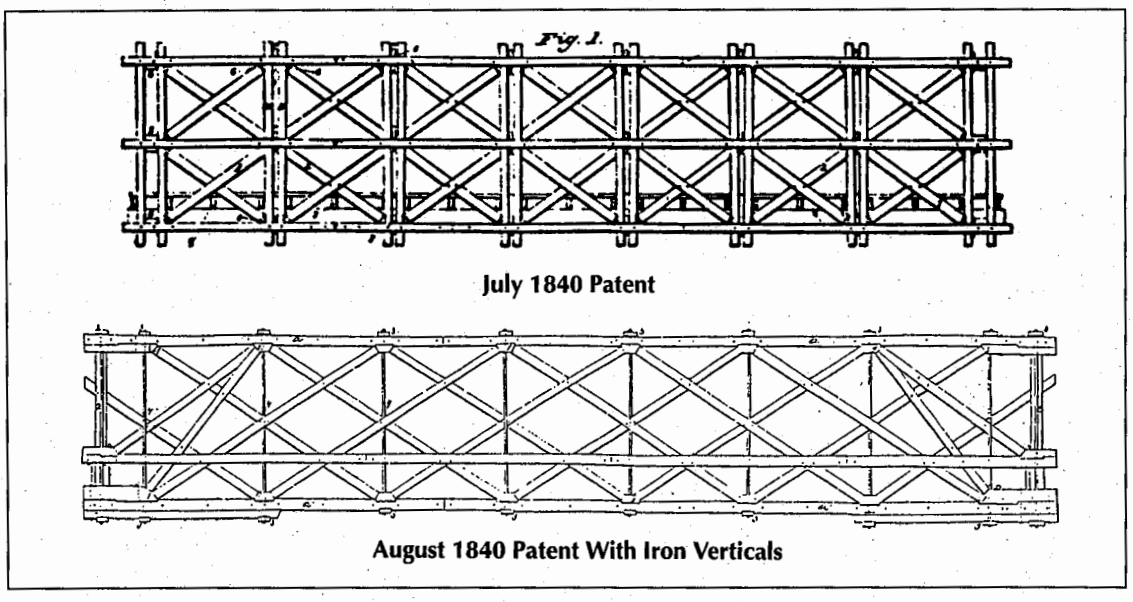

FIGURE 5. Howe's patent for a bridge with a double intersection pattern.

Long never proposed anything other than a truss with a rectangular profile — *i.e.*, the end posts are vertical. His diagonal patterns and characteristic loadings with the diagonals in tension and posts in compression, as well as diagonals in compression and posts in tension, are supposedly the distinguishing characteristics of what are commonly called the Pratt and Howe trusses.

The Howe Truss

In August 1840 William Howe patented a wooden bridge with verticals in iron and with a double intersection pattern of wooden compression diagonals. The previous month Howe had been granted a patent for a similar truss but with wood instead of iron verticals (see Figure 5).[6] It is of interest that Henry Wilton had patented a combination "Town Truss" with a supporting arch that had iron verticals (he called them "suspended braces") in June 1834.[7] He claimed that "the employment of the vertical braces and horizontal bolts" would remove "the difficulty experienced in the construction of the lattice bridge in their twisting and separating which destroys the bridge and which the suspended braces and horizontal bolts above described effectually prevent."

Later, in August 1846, Howe received a patent on a combination truss with wooden diagonals, wooden chords and iron verticals (see Figure 6). The diagonals in this truss only carried over one panel. It included an arch from which he intended to adjust the camber of the truss through "regulating screws that are made to bear on the arch beam."[6] It is not known if he ever built a truss to this patent. It is also not clear whether he could adjust a full-length truss using the regulating screws.

It is strange that the truss shown in Figure 7 that carries Howe's name bears little resemblance to the three patents that were issued to him. His only claim to originality, and that is a weak claim, was the use of iron verticals in tension.

Interestingly enough, Frederick Harbach patented on August 12, 1846, an entirely iron truss in the so-called Howe pattern (no arch) almost two weeks before Howe's last patent.[8] Harbach referred to "Howes patent bridge," which he said consisted of "diagonal braces, abutting blocks and tension rods, the braces act by thrust only." Harbach maintained that the braces "in no respect operate by tension or as suspension braces." Harbach's iron braces and counter braces were connected with iron saddles that, in turn, were connected to the upper and lower iron chords. The top chord was made of cast or wrought iron tubes and the lower chords of riveted wrought iron boiler plate. He

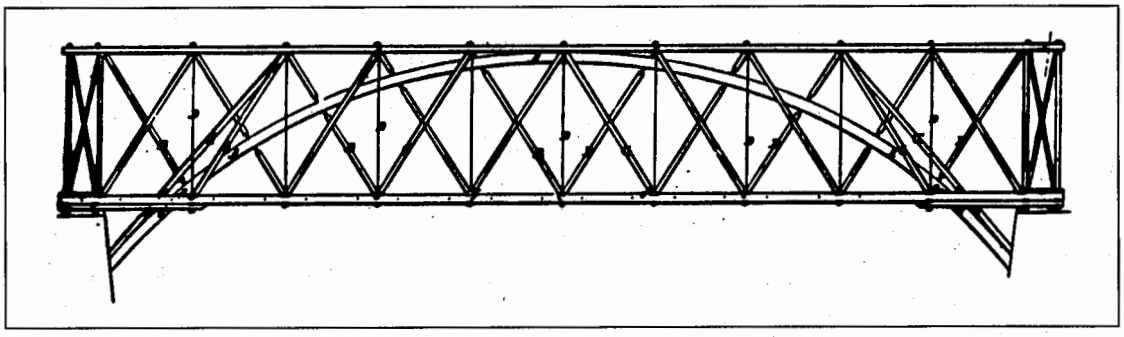

FIGURE 6. Howe's 1846 combination truss bridge.

proudly stated that "by combining with the braces, and counter braces, the toe pieces and with the tension rods the supplementary screws and nuts as above described, I effect a very important improvement in such a bridge truss, as the braces and counterbraces may thus be made to act as suspension or tension, as well as thrust braces, and thus to resist the vertical strains upward as well as downward." In addition, he added nuts on his vertical rods both above and below the upper and lower chord. By so doing his "vertical rods at the same time operate either by tension or thrust."

Harbach built at least one bridge using his patent on the North Adams Branch of the Boston and Albany Railroad just north of Pittsfield, Massachusetts, in the 1840s, as well as reportedly several others in Ohio.

The Pratt Truss

Thomas and Caleb Pratt in 1844 patented a wooden truss with crossed iron diagonals in each panel and with wooden verticals in compression (see Figure 8). Their patent claim was based on "a method of constructing a truss, that is to say the combination of two diagonal tension braces and straining blocks, in each panel of the truss frame of a bridge; by means of which the camber may be regulated so as to increase or to diminish it, either in whole or in sectional part of the bridge."[9] They wrote that:

> *"[T]he bracing by means of tension bars extending diagonally across each panel of a bridge truss has been long known and used* [emphasis added]; but the system of bracing and counterbracing by means of tension bars crossing each other in each panel, is believed to be new, and not only affords the means of regulating the general camber of a bridge but allows it to be drawn up, or depressed, in any particular segment, at pleasure, and thus furnishes a means of regulation not derivable from the single tension braces in each panel."[9]

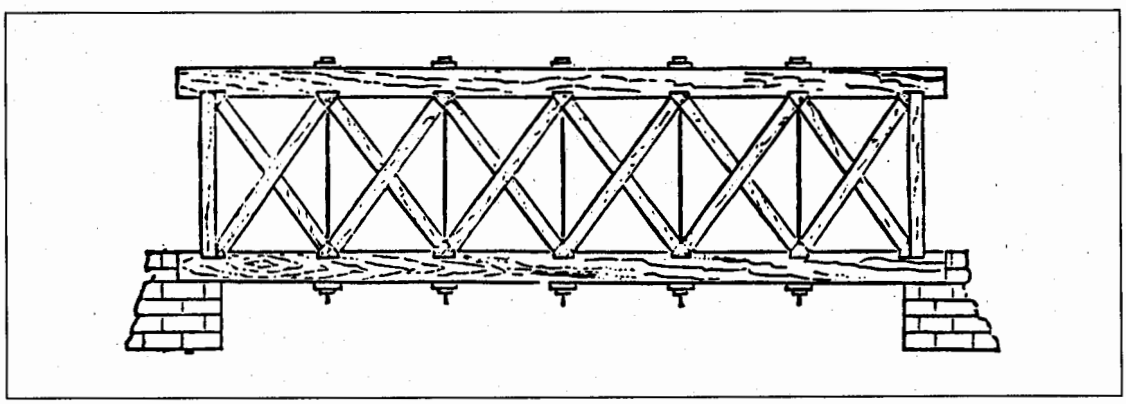

FIGURE 7. The truss commonly referred to as the "Howe truss."

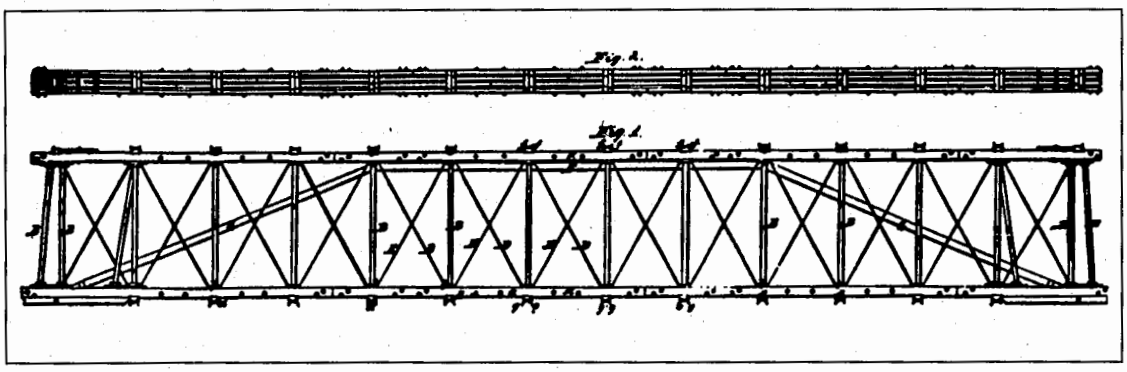

FIGURE 8. Pratt truss patent drawing.

They even wrote that "in some situations where a truss may be employed which has its braces and counterbraces arranged as exhibited in Figure 1 [bottom part of Figure 8 here], the lower stringer may not be requisite as the flooring or whatever is held up by the truss may be depended directly from the lower straining blocks or other convenient parts."[9] This statement shows a complete lack of understanding of the role played by the lower tension chord and was also the reason for the later collapse of several Rider bridges that incorporated this design.

A standard rectangular truss pattern with the end post vertical or near vertical is shown in Figure 8. All of the tension diagonals ran only over one panel. The "arch beam," as it was called, was optional "should it be desirable to increase the strength of the truss."[9]

What then did the Pratts invent? They were wrong about the use of two tension diagonals being new since Long had done the same thing in 1839 (and it will be shown below by Whipple in 1841). Long did use the vertical post to set his camber while the Pratts used nuts on the threaded ends of their iron diagonals, but that is a minor difference. It is also interesting that the so-called Pratt truss that was built later in the century using iron and steel did not have any way in which to adjust the length of the diagonals to maintain the truss in its design configuration without distortion and as such did not use the only new feature of the truss as it was patented.

The Rider Truss

A year and a half later Nathaniel Rider patented a bridge design that reflected, in his words, a "new and useful improvement in bridge-trusses having diagonal braces, all of which are subjected by the forces which usually act upon a bridge to tension strains" (see Figure 9).[10] Rider also did not make any false claims on his application:

> *"I am also aware that in general manner or principle in which the several diagonal tension braces, horizontal top and bottom stringers, and vertical posts are arranged and operate together there is no substantial novelty* [emphasis added]. I do not mean therefore to be understood that my discovery or invention is to be found therein. It is however to a certain extent in an arrangement of the straining power with respect to tension braces and posts, so that it shall not act unequally upon them, or so that the force applied to effect a strain shall apply to two of them at once, and be distributed equally upon each thereof."[10]

Rider had simply taken Long's truss in wood and built it in iron. To Pratt's truss he added a strange cambering device that acted somewhat like Pratt's additional "arch beams" except Pratt's was a compression device under the upper chord while Rider's was a tension device under the lower chord. Notice that Rider also had a truss with a rectangular profile. Rider referenced Pratt's Truss in his application as follows:

> "[T]he tension bars or braces as well as the top and bottom strings . . . I make of wide plate iron, which I refer to rods of iron, such as are used in other bridges of this kind,

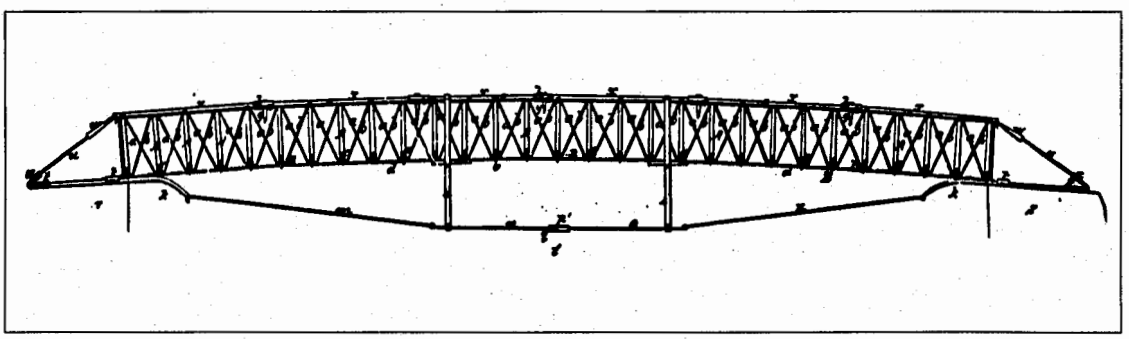

FIGURE 9. Rider truss patent drawing.

particularly in one described in certain Letters Patent numbers 3523 and dated the fourth day of April, A. D. 1844 [Pratt's], my invention being intended as an improvement consisting in wholly dispensing with 'straining blocks,' such as are described in the said Letters Patent."[10]

Long later came to an understanding with Rider in which Rider attributed to Long the original patent idea and apparently made a settlement with him to allow the further construction of Rider bridges. Long (according to Richard S. Allen, a noted wood bridge historian) also went after the Pratts, but with little success since the bridge they designed was never built in wood or iron for some time. The Pratts never pursued Rider even though he had mentioned their patent in his own application.

In late 1846, therefore, Long had laid the basis for diagonals in tension and posts in compression, while Howe introduced iron for posts in tension and wood for diagonals in compression (which was modified to an all-iron truss by Harbach). Long had predated both of them with his 1830 Jackson Bridge. The Pratts had, perhaps unknowingly, taken Long's idea and made their diagonals in iron, which Long had already considered, while Rider used iron for all parts of his truss.

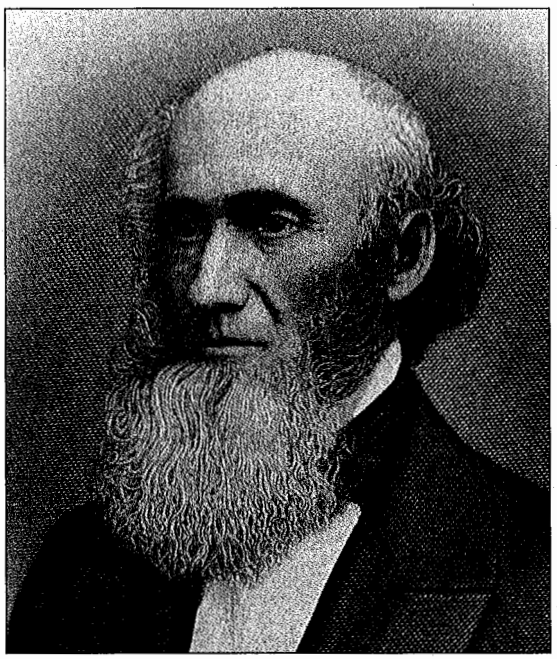

FIGURE 10. Squire Whipple.

Whipple's Bridges

Squire Whipple patented his bowstring arch under the title of "Construction of Iron-Truss Bridges" in April 1841 (see Figure 10). After describing the top chord, thrust tie and verticals for this truss, he wrote that the design would have:

> "[D]iagonal ties . . . of wrought iron, or braces of cast iron, in pairs crossing one another between the vertical rods and between the arch and the thrust ties, except under the end segments . . . of the arch where only one tie or brace, Figs. 1 [top in Figure 11 here], and 3 [second from bottom in Figure 11 here], is used extending horizontally from the end of the arch to the foot of the first vertical or from the end, Fig. 5 [lower right-hand corner in Figure 11 here] . . . The vertical rods may be made of round iron, and in all cases should have an aggregate strength sufficient to sustain the floor and any addi-

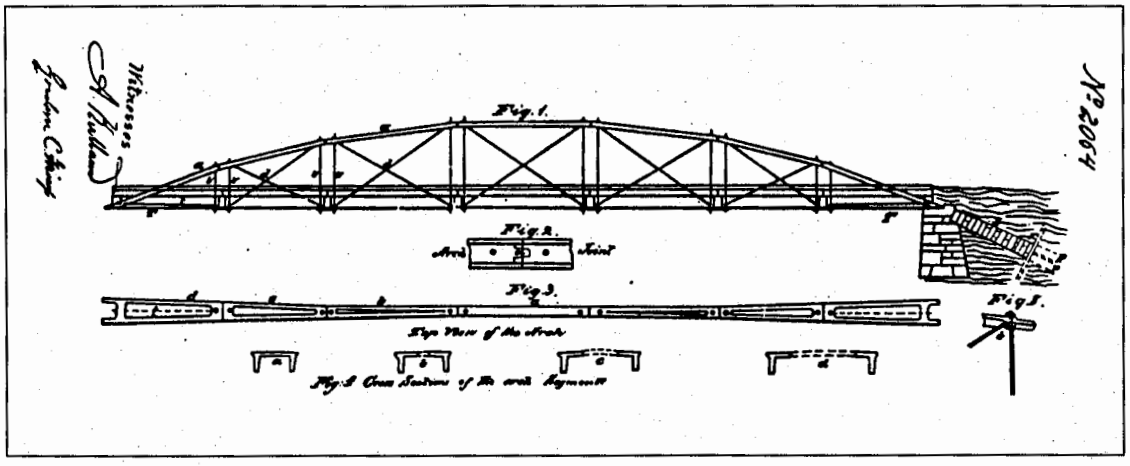

FIGURE 11. Whipple's 1841 patent application drawing.

tional weight that may come thereon, and when the wrought iron diagonal tie is used the vertical rods should have a larger size to give them stiffness as posts. . . . When the cast iron brace is used instead of the wrought iron diagonal tie, the vertical rod is never subjected to a thrust or negative force and may be of a smaller size and furnished only with a bolt head at one end and a screw-nut at the other, like an ordinary bolt. Otherwise, there may be one or more posts of cast or wrought iron used in conjunction with the wrought iron diagonal ties, in which case the vertical rods may be made smaller, or dispensed with entirely, the diagonal ties being enlarged so as to be adequate to sustain the whole weight."[11]

Whipple's patent, unlike any bridge patent before, showed an understanding of the structural behavior of the diagonals and verticals and the need to size the verticals to handle their loads as either a tension or compression member. Once again, like Long, he had crossing diagonals in each panel (except the end ones) in tension with posts (verticals) in compression. It is not evident that he used cast iron diagonals and, thus, never used verticals in tension. Therefore, like Long, he predated both Pratt and Howe in covering the essentials of the latter two patents.

Whipple analyzed various types of truss forms and suggested that some were more efficient than others. He wrote that:

"[P]rior to 1846, or thereabouts, I had regarded the arch formed truss as probably, if not self evidently, the most economical that could be adopted; and at about that time I undertook some investigations and computations with the expectation of being able to demonstrate such to be the fact, but on the contrary the result convinced me that the trapezoidal form, with parallel chords and diagonal members, either with or without verticals, was theoretically more economical *without than with* vertical members — there being shown a less *amount of action* (sum of maximum strains into lengths of respective long members) under a given load."[12]

His study also indicated that:

"[I]t was apparent that each of the three forms — the arch, and the trapezoidal with and without verticals, possessed certain practical advantages entitling each to preference in respective cases, and, no other forms or combinations presenting themselves which seemed capable of competing successfully with these, they were assumed by me as those which would be the prevailing forms which coming practice would adopt."[12]

In 1846 Whipple wrote an essay in which he set forth the basis of scientific bridge building, much of which had been used in his bowstring truss design. This essay was collected with an-

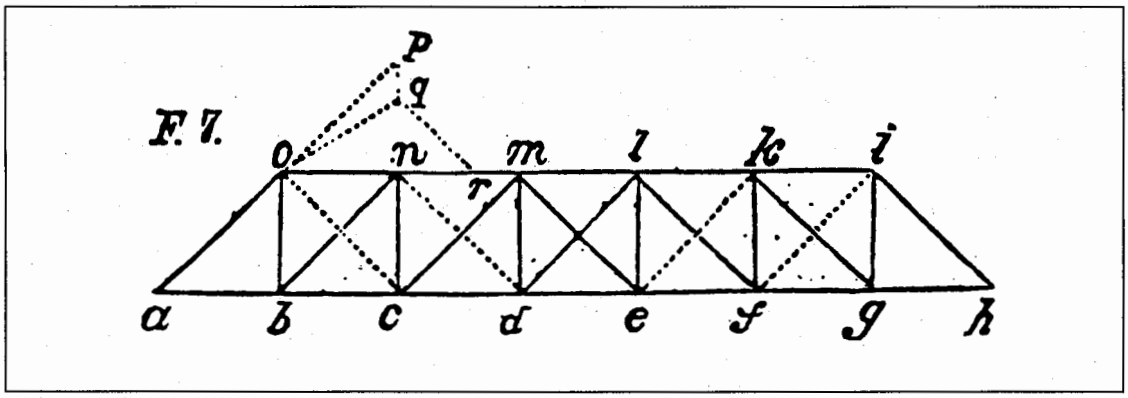

FIGURE 12. The truss Whipple analyzed in his 1847 book.

other into a book the next year. This book, *A Work on Bridge Building Consisting of Two Essays, The One Elementary and General, the other Giving Original Plans and Practical Details for Iron and Wooden Bridges,* was one of the first to set forth the scientific principles of bridge building.[13] He compared his trapezoidal truss with that of Howe and Long (he never mentioned Pratt) as follows:

> "[T]hough the general plans and principles of this truss have been extensively used in bridge building, still, for reasons which I will not, in this place, undertake to explain, the principles have never (to my knowledge or belief) been applied with that skill and economy in the proportions of the different parts of the structure, which are necessary to secure the advantages above pointed out as practicable.
>
> "For instance, the bridges built on the plans generally known as Howe's or Long's plans, are essentially the same in their general features. . . . But instead of being built with each part proportioned to the maximum stress to which it is liable, the upper and lower ribs or stringers have been of the same dimensions throughout, or nearly so; whereas, the stress varies . . . some fifty per cent or one-half. The verticals and diagonals have been nearly of the same size throughout, or increased towards the ends to a far less degree than the comparable stress to which they are subjected in those places, demands."[13]

In that manner Whipple dismissed the work of Howe and Long. Whipple's patent was the first to give rules for sizing members and for spelling out details for a typical 60- to 80-foot truss carrying a load of 100 pounds per square foot of deck area. The size of his cast iron top chord (at mid-span) was described as:

> "[T]he depth about half the width and the thickness of metal about one-tenth of the width, size, form and proportions to be varied according to circumstances or the taste and views of the engineer . . . the remaining segments to be formed and adapted to the preceding in the same manner last described, but increasing in width and divergence to the end of the arch, where the width may be about three times as great as the middle, with the same area in the cross section except at the cross bars connecting the two halves of the segments . . . when the rise of the arch above the thrust tie is from one tenth to one eighth of the length of the span, thrust ties having an aggregate cross section of about one square inch for each hundred square feet of bridge floor will be about a suitable size in ordinary cases."[11]

In his book Whipple came to the realization that the trapezoidal form of a truss with a sloping first top chord member (which he called his end brace) was to his surprise more efficient than his bowstring. He was the first to promote this type of truss. In the first essay in the book he analyzed a truss with all the characteristics that were patented later by the Pratts and are now called Pratt trusses (see Figure 12). In an 1855 article in the *American Railroad Journal,* he

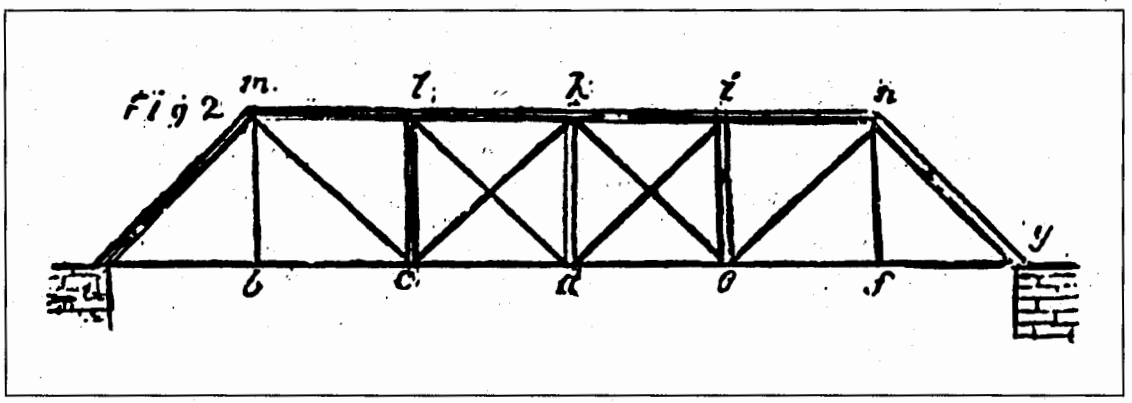

FIGURE 13. The Whipple truss that was shown in the Bollman article.

compared this type of truss (which he said was "arranged and practiced upon by me" and is shown in Figure 13) with the Bollman truss (which he called "a mere fossil of one of Whipple's discarded principles, dug up, and probably considered to be original by Wendell Bollman").[14]

In the January 29 issue of the same journal, Whipple completely analyzed the same truss since he "had been requested by several readers of the Journal to give demonstrations in relation to the maximum strains upon the several parts of the bridge truss."[15] This may have been the first time he had demonstrated, in a journal, the method of analysis he developed in his book.

This truss pattern had single diagonals in tension and counter braces only in the center panels. Whipple had shown in his 1847 book that counter braces only made sense in the panels near the center of the bridge. He wrote (see Figure 12):

> "The counter diagonals *nb, me, lf,* and *lg* can manifestly, only act, when the main diagonals, *cc &c,* to which they are respectively opposed, are relaxed, *i.e., nb* can only act when the tendency of the variable load is to produce a greater action upon this part, than the weight of the structure alone tends to produce on *oc,* and then, will only have an action equal to the excess of the former tendency above the latter . . . . consequently the counter diagonals *nb* and *kg* will usually be of no use in the structure, except for appearance and may be dispensed with. . . . [I]n like manner, the maximum action in *mc* and *lf* will be a negative quantity, and the parts may be dispensed with, especially for common road bridges, which usually are not liable to be exposed to very large variable weights."[13]

This particular type of truss was adopted by John Murphy in the Philadelphia area and became known as the Murphy-Whipple truss (see Figure 14). Murphy, a Rensselaer graduate, worked with Whipple for a short period on the enlarged Erie Canal. Upon moving to Philadelphia, he and a classmate from Rensselaer, George Plympton, built many Whipple trapezoidals and bowstrings in that area giving Whipple his patent fees for the latter and credit for developing the former. Murphy is best known for substituting turned pins for Whipple's loops and cast-iron shoes. He also gradually substituted wrought iron for cast iron in his top chords and verticals. When asked what was the relationship between Murphy and himself, Whipple responded:

> "I first met John W. Murphy about the year 1850 or 1851. I learned that he thought well of my book and my bridges, whence I inferred, of course, that he was a man of discrimination and ability, and as he afterwards talked up iron bridges and Whipple bridges in Pennsylvania, he was of service to the cause and to me. Iron bridges 'took' in Pennsylvania rather better than in New York, and Murphy, with others, formed a partnership for building iron bridges, and purchased my patent (covering the arch

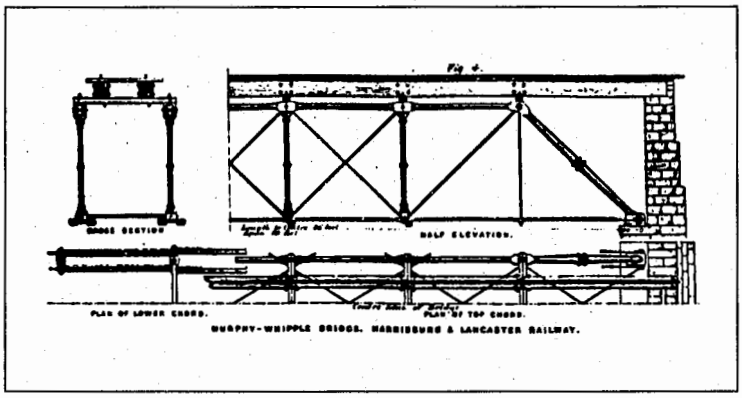

FIGURE 14. The Murphy-Whipple truss as shown in Colburn's paper.

truss only). In the year 1859, or thereabouts, he built a few bridges, which they were pleased to designate as Murphy-Whipple bridges, to which I made no objections, though it has perhaps been the means of disseminating false impressions. 'Murphy-Whipple bridges,' properly considered, simply means bridges built by Murphy upon plans and principles originated by Whipple. My relations with Mr. Murphy were most friendly, and he conceded to me all my claims to originality in the bridge question."[16]

Whipple built iron bridges to this pattern on the New York and Erie Railroad in 1848 and 1849, and on the Utica & Black River Railroad at Boonville, New York, in the early 1850s. He was well aware that Murphy had adopted his idea and that Murphy was more successful in finding railroads that wanted to build iron bridges than he was.

Zerah Colburn in his paper on "American Iron Bridges," published in the *Proceedings of the Institution of Civil Engineers,* shows the bridge pattern and calls it a Murphy-Whipple saying that "the Murphy-Whipple Bridge . . . was believed to be the best class known in the States."[17] C. R. Manners in an *Engineering, London* article, reprinted in the *Scientific American Supplement* March 12, 1892, called the pattern a Whipple truss.[18] A. P. Boller in his 1890 book, entitled *Practical Treatise on the Construction of Iron Highway Bridges for the Use of Town Committees,* designated this pattern as a Whipple single cancelled truss, which is the terminology Whipple used in his book (see Figure 15).[19]

Therefore, in the period from 1850 to 1890, most of the technical literature designated this truss pattern as either Whipple or Murphy-Whipple and not a Pratt. Why and when did a Whipple truss, or the Murphy-Whipple truss, become known as a Pratt truss?

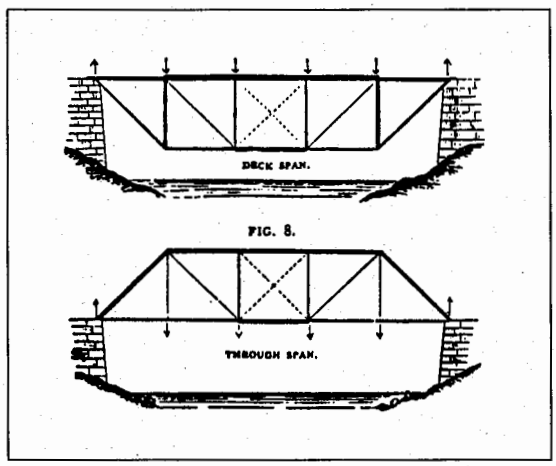

FIGURE 15. Boller's illustrations of Whipple bridges.

The Whipple Double Intersection Truss

In his 1847 book Whipple analyzed a trapezoidal truss for long spans (greater than 120 feet). A long span required a high truss in order to minimize the use of metal. He had shown that the diagonals, to be most effective, should be placed at 45-degree angles. He also knew that if the panel length was too long, the cost of the deck structure would be excessive. In order to meet all these criteria he had his diagonals cross over two panels. This method kept his angle to around 45 degrees. It also kept the panel length down and made the height of truss large.

He presented illustrations of this style of bridge (with verticals for iron trusses and without verticals for wooden trusses), which was similar to one he built across the Mohawk River at Freeman's Ferry near Schenectady, New York

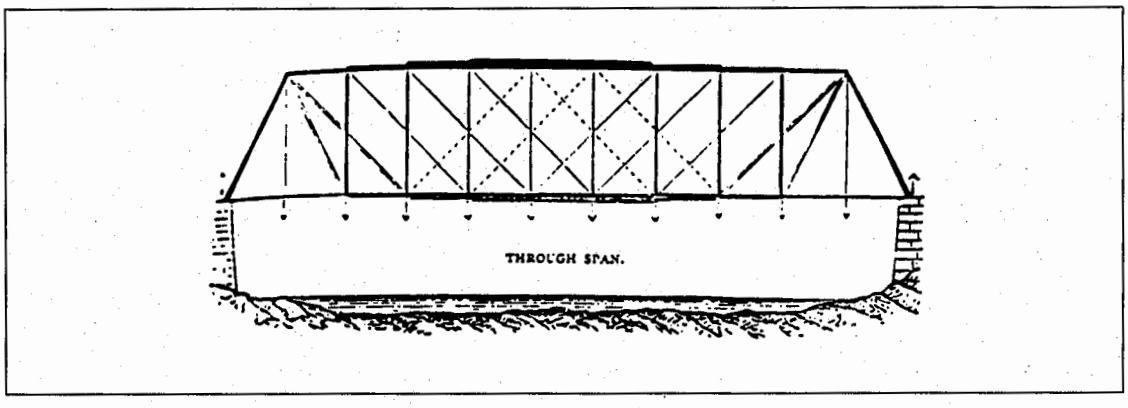

FIGURE 16. The 1847 Whipple double intersection truss.

(see Figure 16). It was a model of this truss in iron that he sent to the American Institute Fair in 1846 and for which he received a silver medal. He also utilized this type of truss for the bridges he built across the enlarged Erie Canal at West Troy and over the Mohawk River at Utica.

Murphy and Jacob H. Linville, like Whipple a Union College graduate, built many bridges to this pattern after the Civil War. In fact, Linville patented the truss (with eye-bar links for the lower chord) in 1862 — over 15 years after Whipple had designed it and over nine years after he had built the West Troy bridge. F. C. Lowthrop had also patented a similar truss pattern. A. P. Boller, George Morison, Charles Macdonald, T. C. Clarke and others built this pattern in steel with spans well over 500 feet. The first American cantilever built by C. Shaler Smith over the Kentucky River in 1876 and the Glasgow Bridge over the Missouri River (the first all-steel bridge) were also built to the Whipple pattern.

Starting around the turn of the century some people began calling the truss pattern a double intersection Pratt. The Pratts never patented, nor built, a truss with a trapezoidal profile. Nor did they ever mention the idea of having tension diagonals span two or more panels. These were all Whipple's ideas, which he had taken and turned into reality by building trusses to the pattern. When did some people start to call this type of truss a double intersection Pratt, and why?

Summary

The Pratts never claimed originality in having crossing diagonals in tension. In their patent application they stated that "the bracing by means of tension bars extending diagonally across each panel of a bridge truss has been long known and used."[9] Long was the first to discuss the use of diagonals in tension and posts in compression in his 1839 patent; Whipple did so later in 1841. Rider paid, or came to some kind of an understanding with, Long for his continued use of trusses with diagonals in tension in the late 1840s. Long also patented a truss with diagonals in compression and posts (verticals) in tension well before Howe received his first patent.

Whipple was the first to propose, analyze, discuss in writing and build a trapezoidal truss in iron with single tension diagonals in each panel and counter braces (ties) only in panels near the center of the truss. He was also the first to design, build and analyze the trapezoidal double intersection truss and recognize its advantages for long spans.

The standard assumption given for calling a truss with a single tension diagonals in each panel with counter braces (ties) in the center panels a Pratt truss is only that Pratt first patented the idea. This assumption is an error. Long was the first and Whipple the second engineer to patent an approach based on this method. Using this rationale, the truss should be called a Long truss.

However, even though Long extended his patent idea to bridges in iron, he never designed or built any bridges in iron, nor did he use single tension diagonals or leave out counter braces (ties) in any panels. In addition, all of his trusses had a rectangular profile.

The only engineer who developed all the characteristics that many people ascribe to Pratt was Whipple. The engineering profession should give credit where credit is due and in the literature designate this type of truss pattern a Whipple trapezoidal truss or a Whipple-Murphy truss for the reasons given.

If the name of Pratt were to be removed from the truss which has borne that name for decades, it would be even easier to remove the name from the double intersection trapezoidal. This truss was uniquely a Whipple design and had its conception as early as 1846. It was used for an actual bridge in wood in 1848 and in iron in 1853.

Merriman and Jacoby, in their textbook *Roofs and Bridges*, wrote:

> "[T]he Whipple truss . . . is an instructive instance of a form which was extensively used from 1850 to 1885, even for the longest spans, but which now is no longer built. This has all the advantages of the Pratt type as regards the use of vertical compression members in the web, and also by the double system of webbing the panel points are brought nearer together."[19]

Waddell, in his book *Bridge Engineering*, wrote:

> "[T]his bridge was of the double-intersection Whipple type; and, on account of the improvements introduced by Murphy, it has frequently been known as the Murphy-Whipple Truss."[21]

Carl Condit, in his book *American Building Art — 19th Century*, included a diagram of a double intersection trapezoidal truss of Whipple's and Murphy's usage and called it Whipple's. Condit noted that this type of truss was widely known as a Whipple truss. He added that Murphy made improvements to the truss, which led to its subsequent widespread adoption for long-span railroad bridges.[22]

It appears that one of the main reasons for growing acceptance of the misnaming of this truss was the bridge poster distributed by the *Historic American Engineering Record*, which designated the truss pattern as a double intersection Pratt.

Recommendations

Appropriate credit should be given to Stephen H. Long and Squire Whipple as the premier bridge engineers of the period between 1840 and the start of the Civil War. Long worked entirely in wood, even though he conceived of designs in iron and was 55 years old when he received his most important patents. Whipple was 35 when he received his patent on the bowstring truss (only one year after Long). Whipple worked primarily in iron, although he did build several major wooden bridges. These two men were the first to apply structural theory to the design of their bridges.

The Pratts and Howe were of the old school of bridge designers. They were trained as carpenters, millwrights and architects — without benefit of a sound scientific education. Thomas Pratt, even though he spent some time at Rensselaer, did not have the ability to design members nor did he size members according to the load placed upon them. Both Howe and the Pratts seemed to have introduced iron only as a means to adjust the camber of their wooden bridges.

Whipple and Long, however, not only understood structural behavior, but were able to design and build trusses that were cost effective. Each of them also shared his thoughts and ideas with colleagues in the scientific literature of the day. Most of Long's work was published in pamphlets and the *Journal of the Franklin Institute*. Whipple published in *Van Nostrand's*, the *American Railroad Journal* and *Appleton's*, as well as in his 1847 book on bridge building.

Whipple was also very aware that some people did not always give him credit for his bridge designs even during his lifetime. Several comments by Whipple follow:

> "'Murphy-Whipple bridges,' properly considered, simply means bridges built by Murphy upon plans and principles originated by Whipple."[23]
>
> "[W]hich the author calls the 'Whipple truss,' and another which he designates as the 'Linville truss.' The former contains an important feature, unlike anything ever constructed, recommended, or even tolerated by me; while the latter is in general features

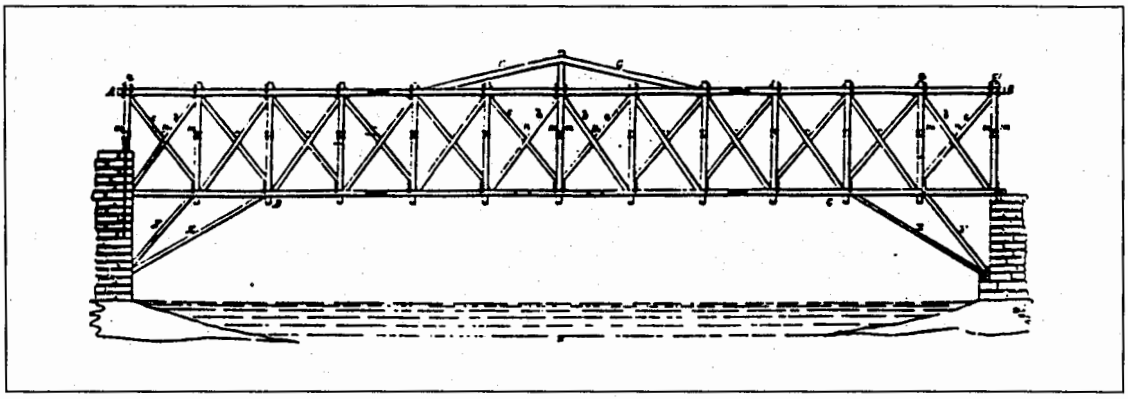

FIGURE 17. The "new" Long truss, replacing the Pratt (rectangular) and Howe trusses.

the same as was used and recommended by me over twenty years ago."[12]

"[T]he isometric and the Post trusses are merely modifications (and not very favorable modifications either) of a type of truss first used and thoroughly discussed by me."[23]

"[A] mere fossil of one of Whipple's discarded principles, dug up, and probably considered to be original by Wendell Bollman."[14]

"The same fate befell three small structures by the same party, of 23 and 26 ft. span, to carry the Newburgh branch over common highways at and near that village. These were wrought-iron skeleton girders upon the triangular plan, such as have since been called Warren girders, and by some regarded as a newly invented combination."[12]

It is clear that Whipple knew that his book and bridges were the basis for many truss patterns that followed. The only recourse he had to receive due credit was by writing occasional letters to the journals of the day. There is also little doubt that he wanted that recognition and felt that he was not treated fairly by his colleagues in this matter. After his death in 1888, it became easier for others to lay claim to his creations and, with the exception of A. P. Boller, few came to his support.

For these reasons, it is well worth examining the historical record to consider whether Whipple and Long have received sufficient credit in the literature for the trusses they developed in the early to mid-19th century. Such an examination might lead one to adopt the following designations of truss types:

- Long truss, replacing the Pratt (rectangular) and Howe trusses (see Figure 17)
- Whipple trapezoidal or Murphy-Whipple truss, replacing the Pratt (see Figure 18)
- Whipple double intersection truss, re-

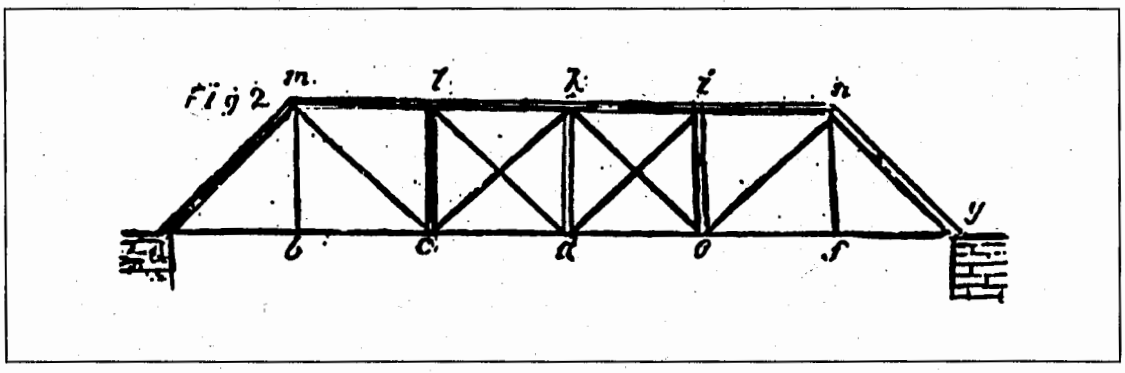

FIGURE 18. The "new" Whipple trapezoidal or Murphy-Whipple truss, replacing the Pratt.

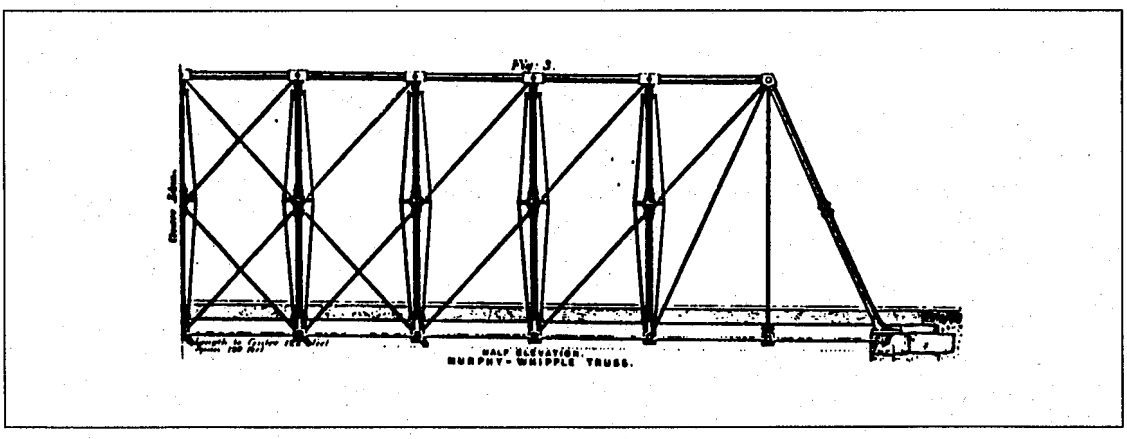

FIGURE 19. The "new" Whipple double intesection truss, replacing the double intersection Pratt.

placing the double intersection Pratt (see Figure 19)

These designations would remove the Pratt and Howe trusses from the roster of truss types in engineering literature. This alteration would be appropriate since their contribution to the field was virtually non-existent. If the Patent Office had examiners with more truss knowledge than they had at the time, they would never have granted patents for work that was not new. This reminds me of a professor who upon returning a student's paper wrote: "Your work is both good and original; unfortunately, the part that is good is not original and the part that is original is not good." Long and Whipple's work was both good and original, and they should be given the recognition they have long deserved. In doing so, a serious lapse in scholarship in the field of history of bridge engineering would be corrected and justice would be done.

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