

REPAIR & REHABILITATION II



Compilation 20

American Concrete Institute

Repair & Rehabilitation II

ACI Compilation 20

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

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Parking Slab Replacement Uses Innovative Geonet Drainage System

Rooftop Plaza Rehabilitation

by Robert Tracy

Concrete slabs are generally very durable, but some environments and conditions of use can lead to accelerated deterioration that sometimes leads to expensive repairs. In one such instance, geosynthetics were used during reconstruction of a relatively new rooftop parking slab on a large office building near Denver, Colorado, that failed prematurely.

Design and construction

The original roofing system consisted of a conventionally reinforced structural slab, a rubberized asphaltic membrane, and a rigid insulation layer sandwiched between two thin protection boards and an unbonded concrete topping slab. This slab served as a wearing surface only and was not part of the structural building system. The primary functions of the various components are:

- The structural slab carries all system live and dead loads.
- The membrane layer prevents leakage into the building.
- The lower protection layer prevents damage to the membrane from additional construction and

isolates the insulation/membrane.

- The insulation layer reduces building heat loss.
- The upper protection layer isolates the insulation from the concrete wearing slab above.
- The unbonded topping is a wearing surface for auto parking.

The concrete topping slab, situated directly over occupied office space, accommodates 750 to 800 vehicles. Maintenance operations considered typical involved joint sealing, minor concrete surface repair, and periodic sweeping and cleaning. Similar systems have been known to provide from 20 to 30 years of low-maintenance service life depending on use, exposure, and certain design features.

The original slab was constructed during the mid-1970s and began showing significant problems within the first three years of service. Early problems developed near drain inlets and consisted of disintegrated concrete encasements (diamonds) around drain basins. By 1979, concern had developed over the integrity of isolated slab areas beyond the drain diamonds and the owner

commissioned a study to determine the specific nature and extent of the problems.

The evaluation

The study of the parking slab included visual observations, materials evaluation, nondestructive testing, and a review of original construction documents and field reports. Results indicated that the principal deterioration mechanism was the saturation of concrete surfacing caused by poor drainage along the interface between concrete topping and insulation. This led to damage by frost action that was so severe that many of the joint seals were rendered useless by the third and fourth winters.

Joint sealant failure caused rapid saturation of many slab areas as water entered the slab-joint system, became entrapped atop the protection layer, and rapidly saturated joint faces, causing progressive damage to adjacent surface areas.

Without subsurface drainage, moisture that penetrated through the joint system could only escape by percolation along the interface between the concrete and the pro-



(left) Original condition of parking slab.
(above) Suitability of the filter/separation layer in the drainage net system was evaluated in a trial installation.

tection layer. This saturated the concrete topping along all joint surfaces for extended periods of time. Scaling from frost damage is most severe on saturated concrete.

The deterioration was progressive and would eventually render the slab unserviceable. An appropriate remedial action plan was needed.

Analyzing the alternatives

Predicting the useful service life and judging the cost effectiveness of various repair schemes proved difficult and inconclusive. Reaching a decision to spend considerable money for repairs with only limited data on useful service life was a major stumbling block to proper consideration of partial repairs. To keep the slab in service, Band-Aid repairs were performed, first during slab investigation and periodically thereafter.

After lengthy deliberation (and several additional years of further deterioration), an updated study helped persuade the building owners that the only practical and predictable solution was to remove and replace the entire wearing surface. This replacement occurred when the

existing slab system was eight years old. By then, large portions of the roof parking lot were closed to traffic because of total slab disintegration.

Recommendation

Design criteria were established for the replacement system based on the data and evaluation from the previous investigation and from further research. Some of the possibilities included:

- Restoration in kind using high-quality concrete.
- Providing insulation board with subsurface drainage features.
- An underdrain system to eliminate subsurface water entrapment.
- Combinations of these systems or concepts.

After lengthy debate, a decision was reached to use a new underdrain system and high-quality concrete for rooftop restoration. Selecting an appropriate underdrain system involved a review of three options: loose drainage gravel, a stabilized drainage aggregate section, or a geotextile separation/filter layer and drainage grid.

Evaluating the underdrain alternatives was more qualitative than quantitative. A number of factors were identified that were considered important for proper system performance. These factors are described in Table 1.

System characteristics were rated one through four, respectively, with a rating of one being very undesirable and four being very desirable. Table 2 shows the values given each characteristic for the alternative systems. The totals indicate the geocomposite system is the most suitable.

After reviewing the available data, including cost factors, the geocomposite drainage system was selected. The system was tested for flow capacities, load characteristics, and unrestrained or low topping/net slip potential. A polyethylene geonet was chosen for the underdrain with a polypropylene woven sheet for a slip sheet and separation layer immediately beneath the concrete (Fig. 1).

Transmissivities were checked at various confining pressures (vertical load) and it was determined that a suitable flow capacity was provided

at all operational load levels. Tests were also run to determine insulation load-deformation characteristics in conjunction with the drainage net system. The first helped define grid lockup potential at the net-insulation interface. Extruded polystyrene board was the rigid system used for roof insulation.

Trial installation

A trial system installation was completed on the actual rooftop to evaluate the suitability of the filter/separation layer between the drainage net and the concrete topping slab. Two filter-fabric materials were tested, and concrete mixes were also varied to determine the effect of different water-cement ratios on cement paste permeation through the filter layer and possible bonding of the filter layer to the topping slab.

The trial indicated that both fabrics were equally effective at reducing paste permeation into the cross-linked polyethylene geonet, but the polypropylene fabric developed minimal bond to the concrete topping, whereas the woven fabric — spun bonded-polyester — adhered tightly and became impregnated with and even embedded into the lower concrete slab. Consequently,

the woven material was specified for the slip sheet/filter fabric material.

System installation

Concrete removal began in early July and proceeded at a rapid pace. Because considerable saturation of the underlying insulation layer was revealed, the insulation was removed and replaced as well. Saturation was so severe at building low points that it was necessary to install a second drainage layer beneath the insulation in roof valleys and along drainlines.

Damage to concrete parapet walls at the base of the slab was also

found at isolated locations and a special closure detail was worked out to tie the new topping slab system to the existing parapet wall without unnecessarily damaging or disturbing the wall, which carried the building facade. Salvaging the existing wall realized considerable cost and time savings, while maintaining the building's outward appearance.

Once the insulation was removed, the membrane was inspected for damage, and spot repairs (on less than 10 percent of the surface) were performed. Exposure of the membrane to the elements during reconstruction was mini-

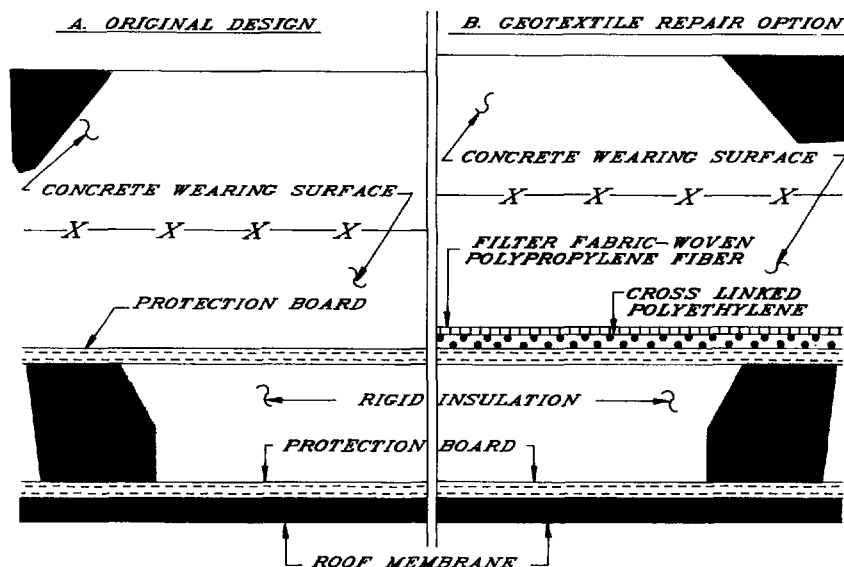


Fig. 1—Schematics comparing original design with geotextile repair option.

Table 1 — Design criteria

Criteria	Description	Value
Load capacity	System must sustain loads without deformation	1000 psf
Reliable drainage	System must free drain under anticipated working loads	2 to 4 gal. per lineal foot of width
Weight	System must apply minimum dead load to existing rooftop	<1 lb/ft ²
Low profile	Add maximum change to roof grades and evaluation	¾ in.
Durability	No significant changes to effective load or flow capacity with time	Corrosion resistant & temperature stable
Restraint	Low potential for lockup between topping & insulation	Low friction coefficient between filter fabrics & drainage net
Ease & speed of construction	Practical application	Manageable logistics; ease and simplicity of installation

Table 2 — Weighted performance features

	Loose gravel	Stabilized gravel	Geonet
Reliable drainage	3	3	3
Suitable weight	1	2	4
Load capacity	4	4	3
Profile	1	2	4
Durability	4	3	3
Restraint	3	2	4
Construction time	2	1	4
Total	18	17	25

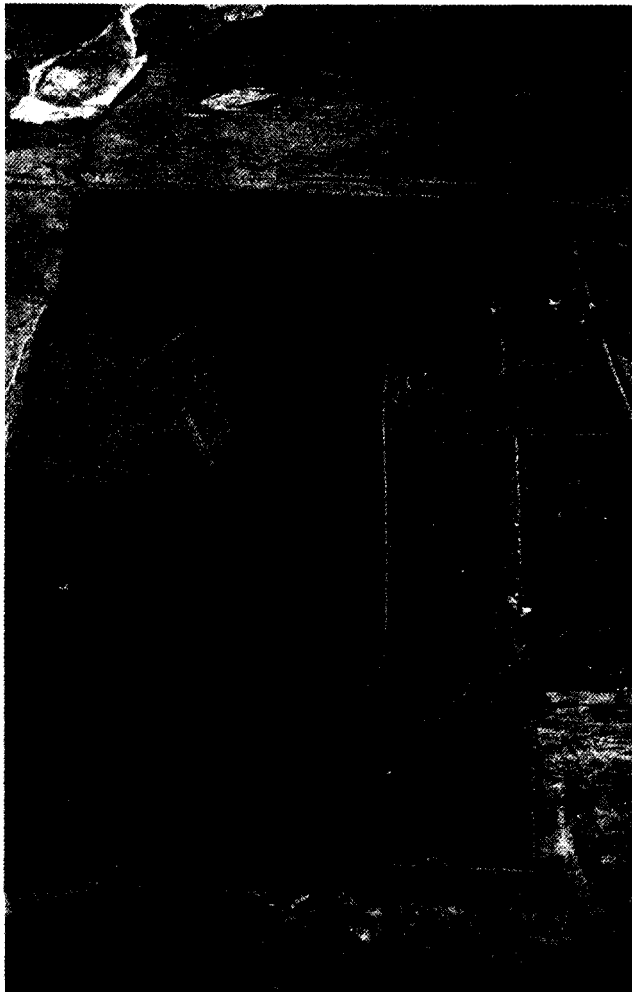


(top left and right) Concrete removal revealed significant damage to the underlying insulation layer.

(bottom left) Spot repairs were made to less than 10 percent of the membrane surface.

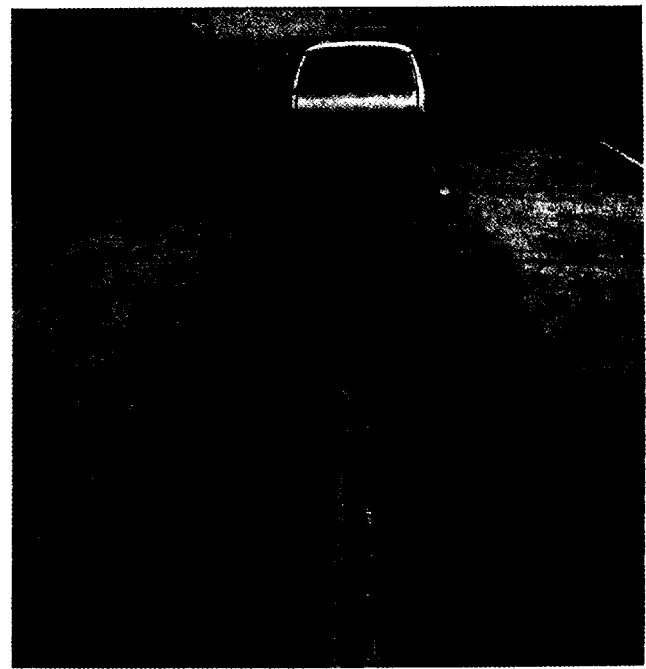
(middle right) The insulation and protection layers and geonet underdrain were placed.

(bottom right) Damage to concrete parapet walls required a special closure detail to tie the new topping slab system to the wall.





Concrete was placed using a strip method to provide proper control joints in the concrete topping.



The finished slab. The system is performing satisfactorily with no reports of operations or maintenance problems.

mized to avoid damage to the building below.

With membrane repairs complete, the insulation and protection layers and geonet underdrain were placed. Formwork was then installed for concrete replacement using a strip method to provide proper control joints in the concrete topping. Strip edges became construction and/or isolation joints depending on the section encountered.

Concrete placement began three to four weeks into the project and a sequenced operation was established to keep removal and replacement on a cycled basis. Special control was used throughout the topping replacement to provide both large-scale movement and crack-control capability.

The building was in continual use during construction and all operations were performed to cause a minimum amount of disruption. Concrete removal, which generated noise and vibration, were performed in the evening hours, and concrete replacement and formwork was done during the day. Weekly progress meetings were held to coordinate operations and resolve any problems that developed. By mid-October, concreting was nearly complete and final cleanup was underway.

Performance

System performance was closely monitored during the first year and, with the exception of some isolated joint-matching problems, sealant problems, and miscellaneous concrete cracking, the system is performing satisfactorily.

As a result of this operation, it can be concluded that geotextile drainage nets, when properly designed and constructed, can provide suitable underdrain layers for plaza decks. With proper design criteria, material selection, and preconstruction testing, these systems can provide a suitable approach for restoring unbonded topping slabs that have undergone premature damage.

This installation firmly establishes that geonet systems can play a significant role in plaza deck restoration, however, although the anticipated performance in the above project has been achieved, it should not be concluded that this system can be used for all applications without further research. Recommendations for further research include:

- Determining slip coefficients for various combinations of geosynthetics over geonet underdrains.
- Long-term deformation characteristics associated with load and temperature.

- Abrasion resistance of geosynthetics systems.
- Shearing resistance to braking vehicle loads.
- Long-term shrinkage and creep aspects of geocomposites.

This installation is considered innovative. Until further design information is developed and research has been performed, the use of this system should only be considered on a project following study and appropriate selection of geosynthetic fabrics. Successes achieved, however, warrant consideration of this system as one viable and cost effective approach to plaza deck restoration.

ACI member **Robert G. Tracy** is principal and founder of Tracy Materials Consultants, Ann Arbor, Michigan. He established this firm to broaden his activities and involvement in restoration engineering. Tracy received both his Bachelor's and Master's degrees in civil engineering from Michigan Technological University. He is a member of ACI Committees 362, Parking Structures; 364, Rehabilitation; and 546, Repair of Concrete.



Repair process developed by Japanese cement company utilizes polymer cement mortar and organic paint film

Repair Method for Salt-Damaged Reinforced Concrete Structures

Salt damage has been an issue in Japan since around 1981. From 1982 to 1984, government organizations and co-operating associations conducted country-wide surveys of salt damage, focusing on railway bridges, road bridges, building structures, and harbor facilities.

A variety of methods are used to inhibit corrosion of reinforcing steel in concrete. When large quantities of external salt are expected to be present, corrosion protection is usually achieved by either preventing penetration of corrosives into the concrete, preventing contact between corrosives and reinforcing steel, or controlling the electric potential of the system. Coating the concrete surface is currently attracting attention as a salt damage repair method.

Asano Refresh (AR) process

The Asano Refresh (AR) process, developed by Nihon Cement Co., Ltd., is a salt damage prevention method that incorporates a concrete surface coating (Fig. 1).

The AR process is a concrete body modification process that improves the durability of existing concrete structures by protecting the concrete against further invasion by harmful substances that can degrade the structure. It can also be applied to a new concrete structures to prevent future degradation.

The combination of materials used in the AR process (Table 1) results in repairs having the following characteristics:

- High adhesivity.
- Protection against the permeation and diffusion of carbon dioxide, oxygen, and water, and against neutralization.



The Asano Refresh process uses prepacked aggregate when a large part of the crosssection must be replaced, as in this salt-damaged offshore pier girder.



Any remaining deteriorated concrete is chipped away prior to beginning the repair.

Table 1 — Main effects of the materials used in the Asano Refresh process

Material	Effect
Corrosion inhibitor (undercoat)	Specially prepared urethane resin (the main component) reacts with rust to inhibit the rust growth. Consequently the surface preparation is only required to a level of 3.
Corrosion inhibitor (finish coat)	
Base strengthening agent	The base strengthening agent offers both the durability of inorganic type and the impregnating ability of organic type. It improves the strength of bare surface weakened by frost damage.
Cross sectional repairing material	The cross sectional repairing material has small drying shrinkage and excellent adhesivity to the base material. No primer is necessary on the base. (Use with SBR latex)
Non-flat preparation material	The non-flat preparation material has small drying shrinkage and excellent adhesivity to the base material. It can also be used to repair a relatively shallow cross sectional chipped part without primer. (Use with SBR Latex)
Surface coating material	The surface coating material has excellent gas permeation resistance, water resistance, crack resistance, and chloride ion permeation resistance. It prevents the invasion of external oxygen, water, chloride, and carbon dioxide. (Use with SBR latex)
Finisher for salt damage prevention (sealer)	The finisher for salt damage prevention is an elastic material to enhance the salt damage prevention of the surface coating material. It has an excellent crack pursuability.
Finisher for salt damage prevention (finish coat)	
Prepacked aggregate injection material	The prepacked aggregate injection material is non-shrink and shows an excellent adhesivity to the base material. It is most suitable for large cross sectional repair at an inverse position. (Use with SBR latex)
Mixed solution	The mixed solution is a styrene-butadiene rubber (SBR) specially formulated for the AR process. By combining with various premix materials, it provides the necessary characteristics.
Injection material	The material is a polymer cement injection material mainly composed of very fine cement particles. It can be injected into fine cracks. (Use with SBR latex)

- Protection against the invasion of chloride ions and salt damage.
- Easy rust removal and rust preventive work on reinforcing steel; high quality rust proofing.
- Stable performance of inorganic materials and specially formulated styrene-butadiene rubber (SBR) latex, which are mixed under strict quality control.
- Economical compared with resin repair processes.

Repair process using prepacked aggregate

Prepacked aggregate grouted with SBR polymer cement mortar is used when a large part of the concrete cross section must be replaced. The accompanying photographs illustrate the steps used to repair a salt-damaged offshore pier girder using this technique.

Remaining deteriorated concrete was chipped away, and sandblasting was used to clean the concrete surface and remove rust from the reinforcing steel. Corrosion inhibitor (primarily a urethane resin) was applied to the reinforcing steel, and enough formwork installed to support the prepacked aggregate. The rest of the formwork was then installed and the grout injected.

After removal of the formwork the surface coat (SBR polymer cement mortar) and finisher acrylic elastic finish material) were applied. The surface coat incorporated a layer of alkali-resistant glass fiber mesh across the joint line between the original concrete and the repair.

Salt damage prevention system

The surface of reinforcing steel in high alkalinity concrete is covered with a passive film that protects the steel against corrosion. However, when chloride ions penetrate the concrete they can destroy the passive film and corrode the steel. The salt damage prevention system of the AR process uses the surface coating and finish materials shown

Table 2 — Water vapor permeability, chloride ion transmission rate, and chloride ion penetration depth of the salt damage prevention system in the AR process

		Sample			Water vapor permeability (mg/cm ² ·day)	Chloride ion transmission rate (μg/cm ² ·day)	Chloride ion penetration (mm)	
		Surface coat (mm)	Sealer coat (g/m ²)	Finish coat (g/m ²)			after 1 month	after 3 month
Salt damage prevention system in the AR process	I	2	100	800	0.87(<1)*	0.35(<1)	0	0
	II	1	100	800	0.86(<2)	0.34(<1)	0	0
	III	1	100	400	1.00(<2)	0.36(<10)	0	0
Concrete body (uncoated)							6.1	10.1

*Objective values in parentheses.

in Fig. 1 to protect the concrete by preventing the invasion of oxygen, water, and chloride.

Following is a discussion of the effects of the materials used in the

salt damage prevention system (polymers, polymer cement mortars, combined organic paint film) and the effect of the system as a whole.

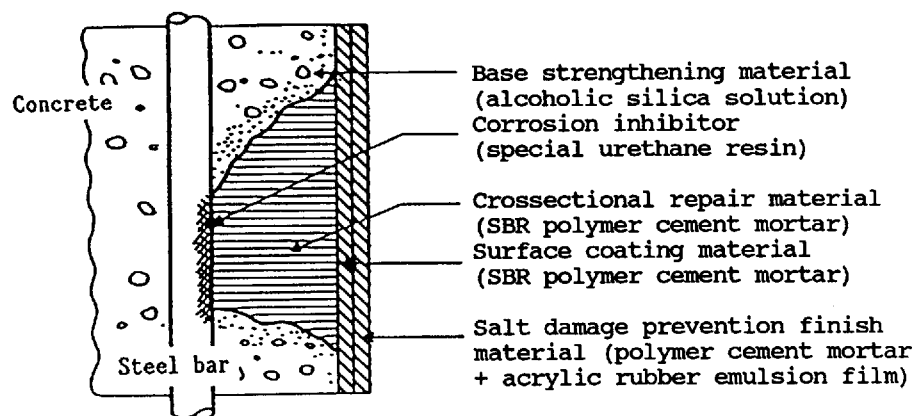


Fig. 1 — Outline of the Asano Refresh process.

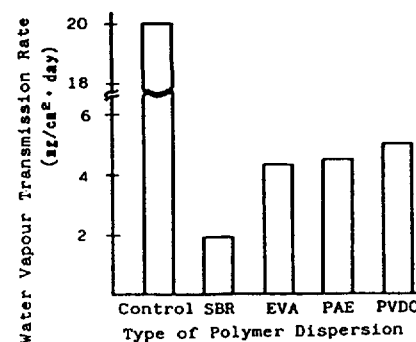


Fig. 3 — Relationship between type of polymer and water vapor transmission rate.

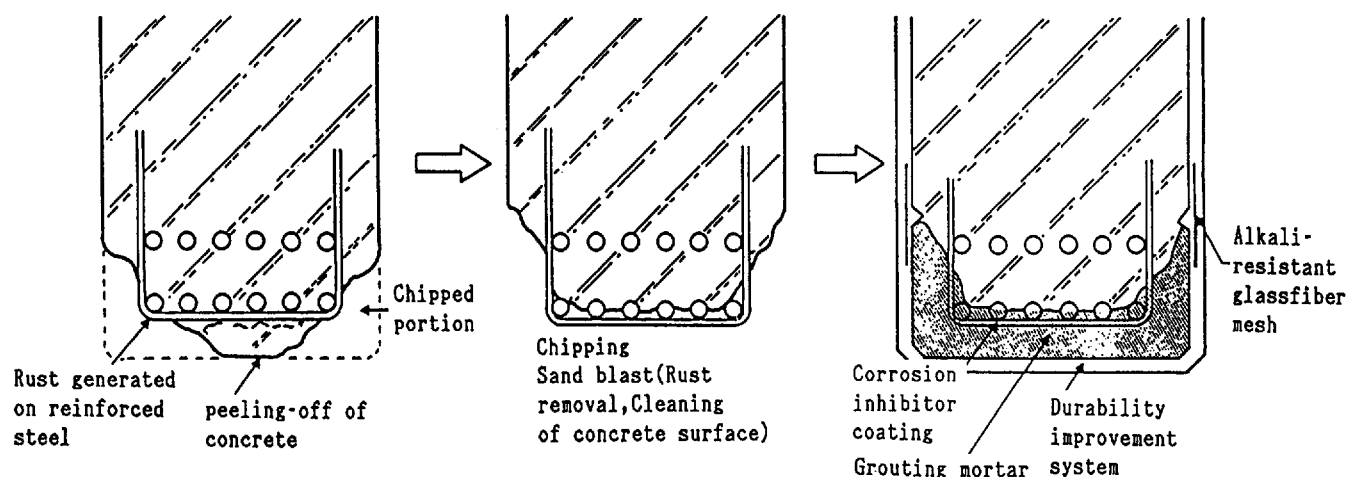


Fig. 2 — Repair procedure using prepacked aggregate.

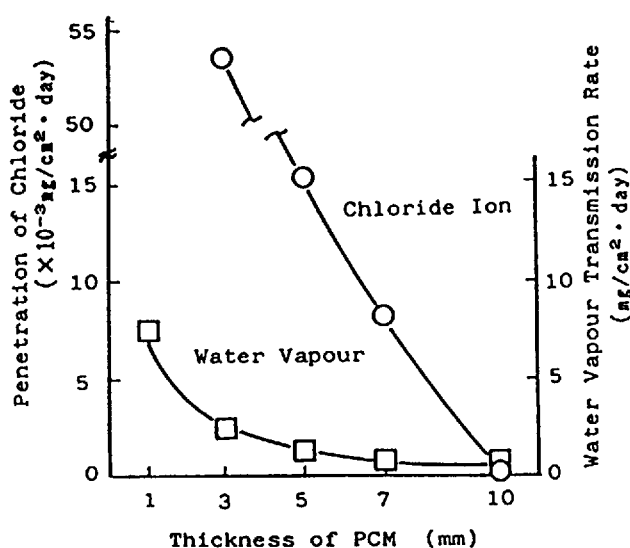


Fig. 4 — Effect of thickness of polymer cement mortar on water vapor transmission rate and chloride ion penetration.

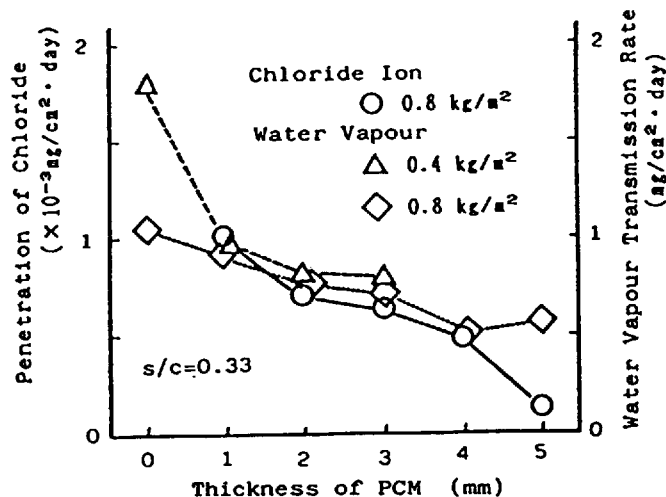


Fig. 5 — Effect of polymer cement mortar combined with organic paint film on water vapor transmission rate and chloride ion penetration.



The concrete surface is cleaned and rust removed from the reinforcing steel by sandblasting.



Corrosion inhibitor, primarily a urethane resin, is applied to the reinforcing steel.



Formwork is partly installed, then aggregate is packed between the formwork and the girder.

Effect of the type of polymer dispersion on water vapor permeation

Water vapor transmission rates were experimentally compared using typical polymer dispersions: styrene-butadiene rubber (SBR), polyacrylic ester (PAE), ethylene vinyl acetate (EVA), and polyvinylidene chloride (PVDC). The results (Fig. 3) indicate that while all polymer dispersions reduce the transmission rate, the SBR group gave a particularly good result.

Effect of the thickness of polymer concrete mortar (PCM) on water vapor permeation and chloride ion penetration

The effect of PCM thickness on water vapor permeation and chloride ion penetration is shown in Fig. 4. A thickness of 3 mm or less did not have much effect on the water transmission rate, but a thickness of 10 mm gave a value of below 1 mg/cm² day, which is the severest target level of salt damage prevention. A PCM thickness of 1 to 5 mm had very little effect on chloride ion penetration, but 10 mm of PCM reduced penetration to approximately 10⁻³ mg/m² day.

In another experiment using SBR PCMs, 6 mm of PCM only slightly reduced chloride ion penetration but 12 mm of PCM completely inhibited penetration. Clearly, thicker layers of PCM can remarkably reduce chloride ion penetration.

Effect on water vapor permeation and chloride ion penetration of PCM combined with organic paint film

The effect on water vapor permeation and chloride ion penetration of combining 1 to 4 mm of PCM with two coating levels (0.4 and 0.8 kg/m²) of an acrylic rubber emulsion film is shown in Fig. 5. (Current formulation could not form a thin coating layer, so this experiment used fine aggregate and the sand-cement ratio was changed from 0.93 to 0.33.)

The ability of the paint film alone to inhibit chloride ion penetration was very low. (The penetration rate of 5 mg/cm² day is outside the

range of Fig. 5.) When the paint was combined with PCM, however, a penetration rate of chloride ion of 10^{-3} mg/cm² day or less could be easily attained with a thickness of only 2 mm. As for the water vapor transmission rate, this did not vary significantly with the thickness of PCM, but the paint film alone gave a transmission rate of 2 mg/cm² day or less. Conclusively, the organic paint film greatly reduces water vapor permeation.

In concrete terms, approximately 10 mm of PCM is needed to satisfy the severest salt damage environment specification. The combination of approximately 2 mm of PCM and 0.8 kg/m² of acrylic rubber emulsion film can form a sufficient salt damage prevention layer, but neither the PCM nor the paint film can provide sufficient protection when used separately at these application rates.

Effect of the salt damage prevention system on water vapor permeation and chloride ion penetration

The results of tests applied to the salt damage prevention system in accordance with JIS Z 0208 (Water Vapor Transmission Test), the chloride ion permeation test (the Japan Road Association method), and the chloride ion penetration depth test (the saturated NaCl solution immersion test), shown in Table 2, indicate that the salt damage prevention system provides relatively low water vapor permeability and low chloride ion penetrability, and largely depresses the chlorine ion transmission rate compared with uncoated concrete.

The water vapor permeability and chloride ion transmission rate necessary for each classification have been studied by the Ministry of Construction and related agencies. However, the values listed in Table 2 are accepted as the most suitable figures at present.

Eiji Sawada is Chief Researcher at the Central Research Laboratory of Nihon Cement Company Ltd., Tokyo, Japan.



Remaining formwork is installed prior to grouting with polymer cement mortar.



Styrene-butadiene rubber (SRB) polymer cement mortar is grouted into the prepackaged aggregate.



Formwork is removed and a surface coat and finisher applied.

Quality Workmanship in Rapid Repair of Concrete Pavements

by R. Frank Carmichael III

States and local agencies are spending millions of dollars annually for the repair and replacement of worn portland cement concrete (PCC) pavements. Much of the money is being spent on improvement and repair projects that require the work to be completed at spot locations in less than 24 hours.

More information is needed on the relative effectiveness of various rapid repair techniques to replace full depth, full lane-width segments of PCC pavement. Data collected by the author on the National Cooperative Highway Research Program (NCHRP) and the data analysis supports the importance of quality workmanship on rapid repair projects.

NCHRP Project 10-24¹ identified, described, and evaluated methods that have been and are being used for rapid replacement of full lane-width segments of continuously reinforced (CRCP), jointed reinforced (JRCP), and jointed non-reinforced (JCP) concrete pavements.

In CRCP, the steel reinforcement is continuous throughout the length of the pavement, and no joints are used except construction joints at the end of a placement. JRCP contains reinforcing steel in the form of deformed steel bars, deformed steel mats, or welded wire mats. JCP is a jointed concrete pavement that contains no steel reinforcement. The joints in JCP or JRCP may be

doweled or undoweled, and sawed or formed.

ARE Inc engineers, in cooperation with various state DOT personnel, established 32 test sites in California, Michigan, Minnesota, Illinois, Virginia, and Texas in 1983 and 1984 to evaluate rapid full depth, full lane-width repairs. State-of-the-art rapid repair techniques and procedures for PCC pavements were made, and the results of 48 months of repair performance evaluations on the 147 patches at the 32 test sites are presented.

Rapid slab replacement techniques have become routine in some states, but in most, only a few experimental installations have been constructed. For example, in some of the largest metropolitan areas with extremely heavy traffic volumes, routine procedures for rapid permanent repairs of PCC pavements have not been developed.

Maintenance engineers in jurisdictions not currently performing rapid slab replacement work indicate a need for it. In some of the states that are currently performing rapid replacements, engineers indicated dissatisfaction with various methods presently being used.

Cast-in-place repairs are the most common approach to the replacement of full depth, full lane-width segments. Since labor costs are generally the most expensive element of the repair cost, cast-in-place methods offer some savings over precast options because of the reduced

number of people required for construction. Also, equipment and hardware used for handling and placing heavy precast slabs are not required for cast-in-place repairs.

However, most cast-in-place repair materials demand longer lane closure times because of curing requirements. Opening the pavement to traffic before repair materials can develop maximum strength shortens the potential service life of the repair.

Precast slabs have not been used much for rapid pavement repairs. Although they offer a potential of shorter lane closure times over many cast-in-place techniques, some agencies have discontinued their use because of high costs and complexities in construction. The higher cost can be attributed to the need for larger construction crews and greater equipment requirements. It is also difficult to establish full support beneath the slabs, which in time start to rock and break under traffic loads.

The design and placement of long-lasting pavement repairs requires a basic understanding of the mechanics involved with pavement behavior and the development of distress in PCC pavements. Little is gained by approaching pavement repair with the philosophy of simply removing the "bad" and re-

Keywords: concrete pavements; evaluation; quality assurance; repairs.

Adjacent Pavement Effects

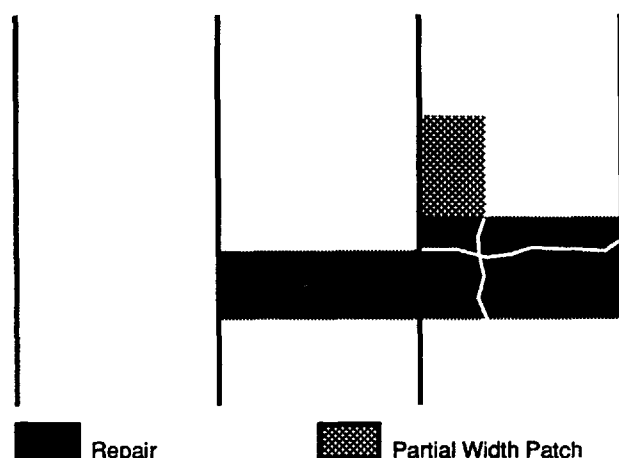


Fig. 1—Example of slab cracking resulting from jackhammer removal methods.

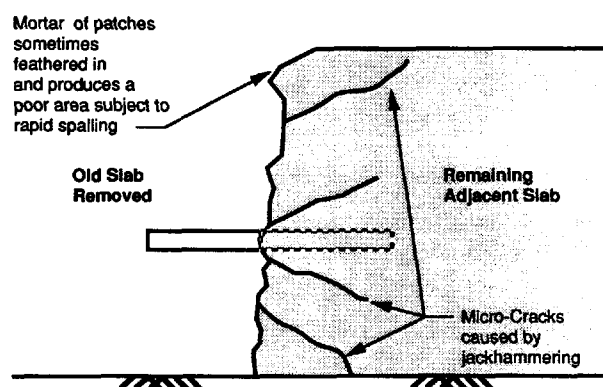


Fig. 2—Effects if repairs in adjacent lane joints do not line up.

placing it with “good.” If the mechanism causing the distress is not also addressed through the repair procedure, the repair normally fails by the same mechanism.

Mistakes also have been made by not recognizing the fact that the pavement structure expands, contracts, and moves in response to traffic and environmental effects. For example, replacement of a deteriorated joint with a patch without a new joint, or placement of a newly formed joint that is not in line with the existing joint in the adjacent pavement, can cause a transverse crack to form in line with the existing joint, which may or may not be detrimental to the repair’s service life.

The summary findings for planning, design, construction, and performance are the result of initial literature surveys and interviews with state and national agencies in 1982, initial site visits by the researchers in 1983, long-term repair performance monitoring from 1983 to 1987 (48 months), and final repair visits and reviews in 1987.

Planning

Most agencies do not use formal procedures to determine the need for a repair. Instead, the decision to repair most often depends on the judgement of an engineer or experienced maintenance personnel. The limits of the repair are also usually established based only on judgement. However, some agencies have

set minimum and maximum repair lengths. The use of agency personnel or contractors for the work is a function of the size of the project.

Based on this study, the findings on repair planning are as follows:

- A very difficult problem in any repair operation is selecting the extent of the area to be repaired. Numerous observations were made where the repair was performing well, but the adjacent area was performing poorly and was disintegrating, which implies that an inadequate amount of the existing pavement was taken out for repair, or that the entire existing pavement section is approaching the end of its usefulness. Nondestructive deflection and radar methods should be used in selecting the extent of the area to be repaired. Such measurements can indicate if repairs are for long-term service, or just temporary measures prior to complete reconstruction.
- Experience in the various states has indicated that the use of precast slabs is feasible, although one state has pointed out that the economics are questionable. Before a final decision can be made as to the use of precast slabs, the total cost, including both construction and user delay cost, should be estimated.
- Experience in several of the states shows that the repair procedure can utilize an assembly line approach, thus expediting the operation and minimizing the cost both to the agency and to the user.

Design

Most agencies design the repair thickness the same as the existing pavement cross-section thickness, and in some cases, a thicker repair is designed. To aid in the design of rapid repairs, mechanistic analysis methods were used to develop design criteria concerning the maximum length of the repair and the time to first saw cut. These analyses include such factors as existing base type, expected temperature drops during construction, and strength properties of the repair material.

Based on this study, the findings on repair design are:

- If the slab placement lengths are too long (> 20 ft [6.1 m]), without an interim joint, random transverse cracking occurs.
- If the slab placement lengths are too short (< 4 ft [1.2 m]), longitudinal cracking occurs.
- Existing steel reinforcement, joint patterns, and joint load transfer details in the repair should be matched to the existing pavement.

The JRCP (jointed reinforced concrete pavement) computer program, developed at the University of Texas at Austin,² was used to estimate the critical state of stress envelopes for a factorial of factors that influence repair performance. These stress envelopes were compared against the expected repair material strength gain at several times for the mean and two confidence levels.

**Table 1 — Maximum placement lengths for repair areas (ft)
(95 percent reliability level).**

Subbase type	Time of placement	15° Temp change		25° Temp change		35° Temp change	
		Type I normal cement	Type III cement	Type I normal cement	Type III cement	Type I normal cement	Type III cement
Rough texture (stabilized)	(Evening)	10	12	7	8	7	8
	(Morning)	18	47	15	29	13	22
Average texture (Asphalt)	(Evening)	24	38	19	30	15	24
	(Morning)	68	120	51	103	43	85
Smooth texture (Visqueen)	(Evening)	36	50	22	35	19	30
	(Morning)	73	120	63	120	57	120

Tables 1 and 2 show the effects of the different factors considered in the analysis on the maximum repair placement length. The two tables represent two different confidence levels that may be interpreted as reliability in a risk analysis. For example, a 95 percent reliability means that for a number of placements, there would be a 5 percent probability that a placement length shown in the table would experience intermediate cracking. A 99 percent reliability implies only a one percent probability of intermediate cracking.

The subbase types in the tables represent the range of possible conditions that may be encountered in the field. The temperature change is the difference between the maximum and minimum concrete temperature during the first 24 hours after placement.

The cement types shown represent a range of materials that may be used. Exotic new cements and accelerators have a faster strength gain during the critical first 24 hours, thus they are expected to allow longer placement lengths. The morning and evening placement implies concrete placed before and after 12 noon, respectively.

A review of Tables 1 and 2 indicates that for the following conditions, shorter slab lengths are required:

- Higher reliability against mid-slab cracking.
- Afternoon placement.
- Greater daily temperature changes.
- Rougher subbase layer texture (i.e., more friction).

Another consideration for reducing the stress that increases with time, and thus preventing random cracking, is to develop criteria for the optimum time to saw slab joints. Using the computer program previously discussed, a stress-time history for the various parameters previously examined was performed. In simple terms, if the stress envelope is above the strength envelope developed by the repair, a crack is predicted to occur.

Thus, sawing should be performed prior to the time the stress

**Table 2 — Maximum placement lengths for repair areas (ft)
(99 percent reliability level).**

Subbase type	Time of placement	15° Temp change		25° Temp change		35° Temp change	
		Type I normal cement	Type III cement	Type I normal cement	Type III cement	Type I normal cement	Type III cement
Rough texture (Stabilized)	(Evening)	8	10	6	8	5	8
	(Morning)	16	38	14	24	13	19
Average texture (Asphalt)	(Evening)	17	33	14	25	13	20
	(Morning)	60	120	46	90	39	72
Smooth texture (Visqueen)	(Evening)	32	44	19	29	15	24
	(Morning)	63	120	52	120	49	120

envelope exceeds the strength envelope. Tables 3 and 4 present the maximum allowable time before sawing is required for a 95 percent reliability and a 99 percent reliability, respectively.

The sequential steps that may be used by the designer to select maximum slab lengths and maximum time of sawing to prevent mid-slab cracking from occurring are given below. The designer must first establish the environmental conditions and constraints, and then use the following procedure step-by-step:

- The first step is to establish a desired reliability; since tables have been provided for the 95 percent and 99 percent reliability levels, one of these must be selected.
- The daily concrete temperature during the first day of construction must be estimated. In the absence of concrete temperature data, the U.S. Weather Bureau air temperature data be used to simulate the

slab temperatures. Since daily temperature change vary significantly during the year, the designer must anticipate what season of the year the slab replacements will be made.

- The next step is to select the representative subbase type.

A *rough interface* would be a cement or asphalt-treated base that has not been rolled to a smooth plane. An aggregate base with large aggregates in the surface would also be in this category.

A *medium friction* condition would be a smooth stabilized base where the surface has been rolled very smooth, and there is little macrotexture in the surface.

A *smooth surface* would be the use of a Visqueen or polyethylene at the interface. However, the application of a Visqueen on top of a very rough surface does not necessarily move it into the smooth category.

- The next decision is to select the time of day the concrete is to be

Table 3 — Maximum allowable time before sawing is required (hours) (95 percent confidence level).

Subbase type	Slab length (ft)	15° Temp change		25° Temp change		35° Temp change	
		Type I normal cement	Type III cement	Type I normal cement	Type III cement	Type I normal cement	Type III cement
Rough texture (Stabilized)	15	NF	4	NS	4	NF	4
	30	NF	4	NF	NF	NF	NF
	60	NF	NF	NF	NF	NF	NF
	120	NF	NF	NF	NF	NF	NF
Average texture (Asphalt)	15	NF	NS	NF	NF	NF	NF
	30	NF	NS	NF	NF	NF	NF
	60	NF	NF	NF	NF	NF	NF
	120	NF	NF	NF	NF	NF	NF
Smooth texture (Visqueen)	15	NS	NS	4	NS	4	4
	30	NS	NS	NF	4	NF	NF
	60	NF	4	NF	NF	NF	NF
	120	NF	NF	NF	NF	NF	NF

NS = Normal Specification Requirement
NF = Not Feasible

placed. For this purpose, the day has basically been divided into two periods: placement before noon (morning) and placement after noon (afternoon). As may be seen (Tables 1 and 2), the maximum allowable spacing differs considerably between these two limits.

- From the appropriate cell in the factorials of Tables 1 and 2, the designer may select the maximum placement length for the prescribed conditions to prevent mid-span cracking. The tables may be used in two ways: using a prescribed length, the designer may select the appropriate cement type to ensure that this length will be satisfactory, or if the designer plans to use a specific cement type, he may establish the maximum permissible placement length.

- After selecting the proper slab length, the designer can use Tables 3 and 4 to select the maximum allowable time before sawing. Again, for the appropriate cell, the designer selects a time wherein all sawing of the transverse joints must be made. If sawing occurs after that time period, it is quite possible that stresses may have built up to a point that random cracking has occurred.

Construction

Construction practices vary widely, but there are several aspects that could adversely affect repair procedures. For example, traffic control is important for this type of construction because the roadway is

usually still serving traffic.

Another important construction procedure is the removal of old pavement. It is typically removed by saw cutting and breaking out by pieces. Removal solely by jackhammering or by using too much jackhammering at the repair edge can cause microcracks in the adjacent pavement (Fig. 1) and ultimately cause poor repair performance, with the adjacent pavement also continuing to crack and spall. Most states require that all loose base material be removed and replaced with new base material or the repair material.

Portland cement concrete with calcium chloride as an accelerator is the most widely used repair material for achieving the fast strength requirements necessary for rapid completion of the job. Curing time ranges from 4 to 8 hours, with some agencies allowing up to 24 hours. Flexural strength in the range of 300 psi to 350 psi (2.1 to 2.4 MPa) is used by some agencies to determine when traffic will be allowed back on the repair.

The possible number of repairs to be placed per day is a function of the construction process and the size of the repairs. As few as five to seven, or as many as 70 to 100 repairs can be placed per day.

While calcium chloride has been the general accelerator of choice, several new products and other repair materials should be examined. These new rapid hardening ce-

ments, polymers, and other materials should be considered and tested. The size of the area to be repaired and the cost and availability of these products should be considered in the decision.

Based on this study, the construction findings are:

- To prevent separation, adjacent lanes should be tied together along the longitudinal joint.
- As pavement is removed, exercise care when replacing existing joints, since stress build up may be excessive if this is not done, and could result in transverse cracking.
- When the continuity of existing steel will not be maintained, the best removal method for preventing damage to the surrounding pavement is to saw the repair limits full-depth and then lift out the pieces to be removed.
- A higher reliability against mid-slab cracking requires shorter slab lengths.
- An afternoon placement requires shorter slab lengths.
- A large expected daily temperature change requires shorter slab lengths.
- A rougher subbase layer texture (i.e. more friction) requires shorter slab lengths.
- Sawing should be accomplished as soon as feasible as long as the blade does not dislodge the aggregates.

The absolute costs for individual activities associated with rapid patching of PCC pavements was difficult to obtain, for a number of reasons. Most construction bid documents lump many activities into composite costs. For example, repair costs may include items such as removal, placement of base, replacement of pavement, dowel bar installation, reinforcement, and curing.

Many rapid repairs also are placed along with other improvements such as repair of longitudinal joints, undersealing, grinding, etc., and some costs are shared, such as traffic control. Also, bid documents, which are the usual source of data, may not reflect total expenditure by item.

Table 4 — Maximum time before sawing is required (hours) (99 percent confidence level).

Subbase type	Slab length (ft)	15° Temp change		25° Temp change		35° Temp change	
		Type I normal cement	Type III cement	Type I normal cement	Type III cement	Type I normal cement	Type III cement
Rough texture (Stabilized)	15	NF	4	NF	4	NF	4
	30	NF	4	NF	NF	NF	NF
	60	NF	NF	NF	NF	NF	NF
	120	NF	NF	NF	NF	NF	NF
Average texture (Asphalt)	15	NF	NS	NF	NS	NF	4
	30	NF	4	NF	NF	NF	NF
	60	NF	NF	NF	NF	NF	NF
	120	NF	NF	NF	NF	NF	NF
Smooth texture (Visqueen)	15	NS	NS	4	NS	4	4
	30	NS	NS	NF	4	NF	NF
	60	NF	4	NF	NF	NF	NF
	120	NF	NF	NF	NF	NF	NF

NS = Normal Specification Requirement
NF = Not Feasible

Table 5 shows cost data collected from participating states. Inspection shows that smaller projects have higher unit costs (Virginia Site 2, Michigan Site 5, Minnesota Site 4, and California Site 3). However, the cost of traffic control is not necessarily affected by the size of the job. For example, Virginia's Site 2 has 5.5 times as many cubic yards of patching, but the traffic delay cost is only 63 percent of the Site 1 cost. This may result from the specific site conditions or bid distribution practices by the contractor.

In general, the cost per square yard of patching ranged from \$50-\$500/yd². Data collected indicates a figure of approximately \$75/yd² as the best average cost estimate, which is calculated by dropping the three highest values and averaging the data.

Repair performance

Most agencies do not have formal procedures for monitoring their repairs, however one agency does have a formal procedure involving condition survey ratings on their repairs. In general, the repair performance is monitored by periodic observation. The performance of the repairs under study in this project were monitored by highway department personnel throughout the duration of the project. ARE Inc personnel made the first and the last ratings.

Subjective reviews of cracking and other distress ratings were extremely useful in developing these results. The author used photographs and crack mapping methods to monitor repair performance. Table 6 shows the criteria found from this study for determining the limits of the repair, or when an existing repair needs replacement to preserve the integrity of the surrounding pavement.

Conclusion

The results from this long-term performance monitoring of rapid placed (< 24 hours) full depth, full lane-width PCC repairs provides a number of conclusive statements:

- Detailed distress surveys and

Table 5 — Unit costs per square yard for rapid PCC repairs.

State-Site#	Area (y ²)	1 Cost of traffic control (\$1,000s)	2 Cost of removal & replacement (\$1,000s)	1 + 2 Total cost of patching (\$1,000s)	Cost of rapid repair (\$/y ²)	Total contract amount* (\$1,000s)
Virginia						
Site 1	572	19	292	311	544	387
Site 2	3,132	12	408	420	134	554
Site 3	8,214	62	951	1,013	123	1,274
Site 4	9,323	111	1,006	1,117	120	1,620
Site 5		No unit prices available				
Michigan						
Site 1	28,044	17	1,455	1,472	52	2,116
Site 2	5,867	10	284	294	50	300
Site 3		No unit prices available				
Site 4	25,481	44	1,200	1,244	49	1,279
Site 5	4,197	7	270	277	66	370
Site 6	26,266	14	1,305	1,319	50	2,049
Minnesota						
Site 1		No unit prices available				
Site 2	2,410	1	178	179	74	192
Site 3	231	7	73	80	346	609
Site 4	75	7	128	135	1800	148
California						
Site 1	1,935	15	93	108	56	110
Site 2	1,900	16	57	73	38	106
Site 3	800	8	88	96	120	189
Site 4	847	7	36	43	51	45
Site 5	1,265	32	101	133	105	413
Site 6		No unit prices available				

*Includes other activities

physical measurements of deflection and/or radar for void detection should be used to establish the limits of each repair.

- Full depth sawing and removal by lifting are the most efficient and effective method of removing the existing pavement.

- Unstable, or poor subbase or support material should be completely removed down to good material.

- Minimum repair length should be 4 ft (1.2 m) and maximum repair length should be 20 ft (6.1 m), or an interim joint or reinforcement

Table 6 — Criteria for Pavement Repair.

	Warning (poor condition)	Replacement needed
Surface deterioration	Loss of coarse aggregate on less than 26 percent of surface area Some coarse aggregate exposed on less than 50 percent of surface area	Loss of coarse aggregate on more than 26 percent of surface area Some coarse aggregate exposed in over 50 percent of surface area
Longitudinal cracks	One crack	Two or more cracks
Transverse cracks	Two or three cracks	Four or more cracks
Number of cracks spalled ¹	Two or less cracks of minor severity	More than two cracks of minor severity or one severe crack
Number of Punchouts ²	Two or less punchouts of minor severity	More than two punchouts of minor severity or one severe punchout
Pumping ³	Minor	Severe
Longitudinal repair edges	None	Poor condition when open crack, spalling, or faulting
Transverse repair edges or joints - Faulted ⁴ Spalled ⁴ Sealed	If less than 1/2 in. high If less than 1 in. wide If partially sealed	If greater than 1/2 in. high If greater than 1 in. wide If not sealed
The following criteria define different possible conditions a slab can exhibit when it shows its first sign of poor condition and when it should be considered a high candidate for replacement. These are suggested values generated from the expertise of the researchers and from interviews with other highway engineers. ¹Number of Cracks Spalled: Minor - less than 1 in. wide Severe - greater than 1 in. wide Note: Both minor and severe crack spalling may be observed. Count longitudinal and transverse crack spalling together. ²Number of Punchouts: () N.O. - none observed Minor - closed cracks no movement Severe - Open cracks, rocking, pieces of pavement missing ³Pumping: () N.O. - none observed Minor - visible stains and fines Severe - deposits of materials on the shoulder ⁴Transverse Repair Edges: Rate only the transverse edges. If a transverse edge of the repair is also a joint, rate only as a transverse edge. For the repair, rate only the transverse edges that are physically part of the repair (i.e., do not rate the transverse edge of the upstream/downstream adjacent pavement). When rating the upstream/downstream adjacent pavement, rate only the edges next to the repair and do not include the transverse edge. Faulting - Warning if less than 1/2 in. Failed if greater than 1/2 in. Spalling - Warning if less than 1 in. wide Failed if greater than 1 in. wide		

should be considered (except for CRCP pavement) to avoid transverse pavement cracking from environmental and traffic load stresses. Old joint spacing should be matched.

- If an old joint is removed, the new repair should replace the removed joint, so that the existing joint spacing is reestablished.
- All reestablished joints should have load transfer devices to prevent faulting or tilting.
- All steel in CRCP should be re-tied and the repair should reestablish longitudinal steel continuity.
- Adjacent lanes should be tied together.
- No significant benefit was noted for the use of the inverted T-repair,

and there was some evidence that the T-wings may easily break off, thus its use is not recommended. It also requires more labor and is difficult to cast and vibrate well.

- Repairs made with Type III cement with an accelerator added can be opened in 6 to 8 hours after placement. Our review did not disclose any agency that was routinely using exotic concrete mixes or new materials, such as polymers, for large repairs.
- Repairs in adjacent lanes should line up so that the joint or end of a repair in one lane is not located mid-panel or offset from the joint in the adjacent lane (Fig. 2).
- Full depth, full lane-width repairs should not be abutted up to partial-width repairs since these

tend to cause longitudinal cracks in the full width repair (Fig. 2).

By adding these simple conclusive findings into the development of a rapid repair design and construction specification, an agency can be assured of the best performing rapid repair. Other useful industry publications are available,^{3,4,5} and most state DOTs are developing recommendations, experience, and specifications in this area. Quality workmanship in the planning, design, and construction will produce repairs that are good looking and perform as expected.

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Ontario Power Generating Station Headworks Rehabilitation

by N. P. Bada

The headworks of the Ontario Power Generating Station is located about 1.5 km upstream from the Horseshoe Falls, Niagara Falls, Ontario, Canada. The headwork structures provide water to the 100 megawatt generation station that was built in the early 1900s at the foot of the falls.

From time to time, minor repairs had been done to maintain the station's operating efficiency and overall safety, but the intake structures had reached a condition where station safety and reliability could no longer be assured.

The gates could no longer be reliably closed completely on demand. Underwater diving inspections during scheduled station outages revealed accelerated concrete deterioration on the maingate apron. The operations staff monitored an increase in loss of generation, loss of forebay water elevation, and loss of water supply to the Niagara Parks Commission Dufferin Island Park. A major rehabilitation program was required.

The rehabilitation program consisted of constructing and removing a 183 m (600 ft) cofferdam along an existing ice diverter on the Niagara River; dewatering the inner and outer forebays; structurally upgrading the ice diverter, outer forebay

spillway, the gatehouse apron, and maingate sills; building new trash racks; demolishing an old screen-house structure; constructing a scenic walkway, and cleaning debris from the forebay.

Cofferdam and dewatering

Constructing the 183 m (600 ft) cofferdam along the ice diverter was the most difficult and challenging part of the entire project. The construction and potential safety hazards at times seemed unmanageable.

The cofferdam was built along an ice diverter within the accelerating water channel immediately downstream from the control gates of the International Control Dam located on the Niagara River (Fig. 1). Although a detailed procedure for installing the cofferdam was prepared, several construction anomalies existed.

These included water currents within the accelerating channel, water flow through the well fractured bedrock, and structural integrity of the ice diverter piers (the concrete condition survey revealed extensive cracking due to ice damage over 80 years of service). There also were concerns about back water pressure on the cofferdam during construction because water levels fluctuate

by up to 1 m (3.3 ft) on the Niagara River to enable two 13.7 m (45 ft) diameter tunnels upstream to provide water to a reservoir in the evening for another generating station.

The water flow through the training wall, limited control of the gates at the International Control Dam due to the International Agreement between Canada and the U.S.A. for providing maximum water flows over the Horseshoe Falls during the tourist season, and minimum environmental impact during the construction and removal of the cofferdam had also to be considered.

The solution was to rest EFCO concrete form panels against a toe beam and header beam, sectionally bolted together underwater by commercial divers, reacting onto individually post-tensioned ice diverter piers and positioned to allow tremie concrete to be placed between the bedrock and the underside of the toe beam (Fig. 2).

Installing the cofferdam

The main cross section of the cofferdam consisted of a W21X55 sill beam positioned horizontally and anchored to react against a bent steel plate assembly that was designed to allow for any misalignment caused by localized concrete deterioration at the pier face. The

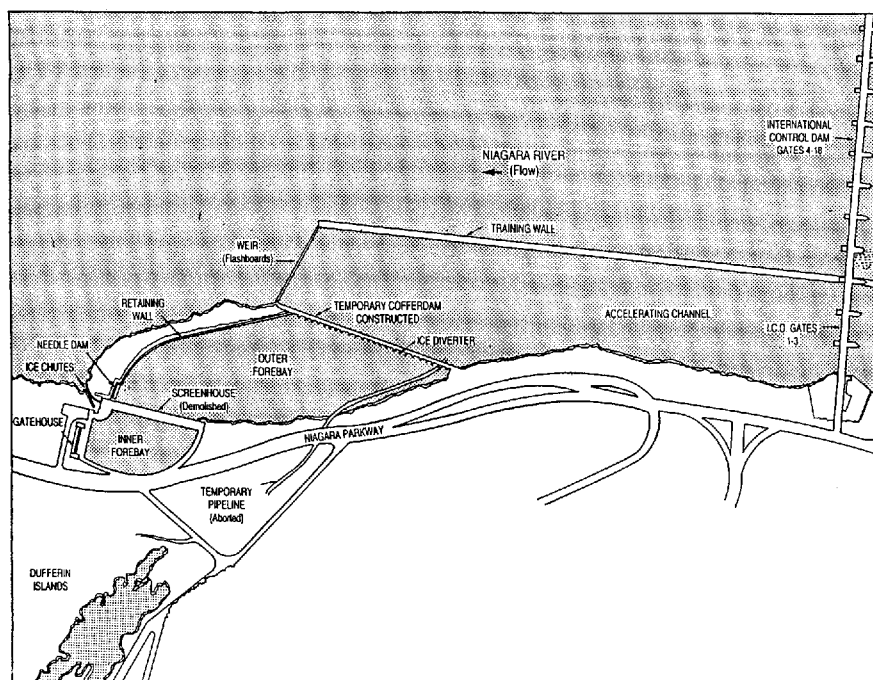


Fig. 1—Ontario Power Generating Station construction site.



Fig. 2—Backside view of cofferdam construction.

space between the steel plate and the concrete pier was filled with tremie concrete. A W12X53 header beam was also positioned horizontally and anchored against the continuous concrete wall of the ice diverter.

The panels (12RX6X10 in.) were positioned vertically and rested on an angle that was welded to the outer flange of the sill beam. Tremie concrete was then placed between the sill beam and the bedrock surface.

The entire operation was performed by commercial divers with two sectional barges (one with a 45 ton conventional crane) on the downstream side of the ice diverter and additional manpower atop the ice diverter.

Installing the bent steel plate assembly and tremie concrete within the space between the pier and the assembly was performed first at all 24 piers.

The sill and header beams were then installed and each section bolted to the next. The elevation of the sill beam was determined by allowing at least 0.5 m (1.6 ft) of cofferdam freeboard at the highest recorded water elevation on the Niagara River. Debris found between the sill beam and the bedrock was then vacuumed and removed. Burlap bags filled with concrete were used

to build a wall on either side of the sill beam to contain the tremie concrete.

Tremie concrete was then placed to the web elevation of the sill beam. Beginning at the upstream side of the river flow, the panels were then installed and continuously bolted and secured back to the ice diverter using wire rope cable.

Because the divers reported a difference in current flows once 80 percent of the EFCO panels were installed, a special gate assembly was designed to accept the last sections of panels. The panel was positioned and allowed to slide down within a gain arrangement.

A 1.2 m (4 ft) diameter valve was designed within a panel to maintain water within the forebay so as not to bottom out the barge. The crane-mounted barge was then towed to shore.

Dewatering

Precautions were taken to insure against an unexpected ice diverter failure once the last panel was securely positioned. Dewatering began almost immediately. A noticeable 0.6 m (2 ft) head difference existed between the upstream and downstream sides of the cofferdam.

The valve in the panel was then closed and the main gates to the

generating station fully opened. Within four to five hours, the 10 hectare area of outer and inner forebays were completely dewatered. Leakage from the cofferdam was minimal and manageable.

No one was allowed to enter the work area until the cofferdam was deemed perfectly safe. A written procedure had been prepared prior to building the cofferdam detailing entry and evacuation of personnel, monitoring the cofferdam during construction restoration, and watering up of outer and inner forebays.

During a routine cofferdam inspection, a tradesman observed a "blow-out" condition. The repairs were made immediately by pumping tremie concrete and thus securing the cofferdam.

The cofferdam was designed to withstand a 7.6 m (25 ft) head of water; it actually held back a water head of 5.2 m (17 ft). The cofferdam installation took approximately 21 ten-hour working days to complete.

Rehabilitation of structures

Concrete mix proportions

Separate mix proportions were submitted to the concrete ready-mix supplier for the tremie mix: 30 25

and 20 MPa (4350, 3625, and 2900 psi) waterline concretes; grout mix; and low control strength material at the time of tendering. Table 1 shows quantity details for the different project locations.

Table 2 shows test results for the various locations. On average, the strength results exceeded their design requirements at 28 days.

Gatehouse pier extension and base slab/batterwall

The gatehouse houses headgates and associated mechanical equipment. Two 5.5 m (18 ft) diameter conduits convey water approximately 2 km (1.25 miles) to the generating station (Fig. 3).

Leakage around the conduits was noticed some time ago and a monitoring program prepared. Underwater inspections indicated that parts of the gatehouse substructures were deteriorating rapidly. Complete closing of the gates on demand could no longer be assured.

The base slab and adjoining bedrock were demolished using equipment adaptable to hydraulic demolition hammers. The base slab was excavated to a depth of 1.5 m (5 ft).

Concrete for the new base slab and pier extensions was placed as a monolithic reinforced concrete slab with no visible cracking. The base slab was post-tensioned using 50 mm (2 in.) diameter hollow-core rock anchors and grouted.

Once the base slab concrete was completed, scaffolding was erected at each pier location. The old concrete surface was bush-hammered, and 25 mm (1 in.) reinforcing bar dowels were installed at 0.5 m (1.6 ft) spacing. The 12.5 m (41 ft) high trash rack gain assembly was accurately positioned at all pier locations.



Fig. 3—Gatehouse repairs. Batterwall is at left.

Table 1 — Quantity sheet

Location	Rock excavation (concrete demolition), m ³	50 mmØ hollow core rock anchors (25 mmØ dowels), each	Reinforcing bars, Mg	Concrete, formed area, m ²	Concrete (CLSM), m ³	Total direct manhours
Gatehouse baseslab, piers, and batterwall	300	28 (320)	80	650	520	23,200
Screenhouse	(1530)	—	6.7	—	165	4000
Needledam/ice chutes	—	(840)	20	238	290 (850)	3500
Retaining (spill) wall	—	146 (4080)	77	407	635	16,750
Install/remove cofferdam (includes fabrication)	—	24	—	—	1000	18,000

Reinforcing bars were positioned and concrete placed monolithically at each pier location. The batterwall was also post-tensioned and concrete placed monolithically. A total of 482 m³ (630 yd³) of concrete was placed over 60 working days of two-10 hour shifts.

Retaining wall

A 215 m (705 ft) long retaining wall, which contains and controls the headworks of the Ontario Power Generating Station, was found in very poor condition. An hydraulic crawler drill was used to drill through the concrete and bedrock.

It was decided to dowel/rock anchor/grout to the bedrock elevation and to cap the spillway with 0.75 m

(2.5 ft) of concrete. A total of 146 50-mm (2-in.) diameter hollow core rock anchors and 4080 25-mm (1-in.) reinforcing bar dowels (candy-canes) were installed.

It was decided to leave the old concrete as a filler and to perform as one side of the concrete form as time did not permit the demolition and complete rebuilding of the wall. Wall sections were formed at 9 m (30 ft) lengths along the wall. Keyways and bulkheads were placed at either end. Filler concrete sections were placed after two days.

A total of 635 m³ (830 yd³) of concrete was placed over a period of 50 days of two 10 hour shifts.

Screenhouse

The original plan was to demolish the superstructure of the screen-

Table 2—Concrete specimen strength tests summary

Section of work	Test no.	Compressive strength, MPa				Design strength, MPa	Air, percent	Slump, mm	Temperature, C	Density, kg/m ³
		7-day	28-day	28-day	28-day average					
Gatehouse exterior intake slab	1	23.8	37.8	37.2	37.5	20.0	5.2	90	17	2,398
Tremie concrete, bottom of cofferdam, 1st 150 LF		36.4 4D (32.5)			46.1					2,330
Screenhouse ice chute 'B' plugs	2	27.6	36.6	37.5	37.1	20.0	5.0	90	18	2,380
Outer forebay, spill wall 21, 19, & 17	3	28.6	38.2	39.0	38.6	30.0	6.4	100	15	2,330
Gatehouse exterior piers 6 & 8	4	27.6	35.9	36.1	36.0	30.0	7.0	115	21	2,287
Outer forebay spill wall #22	5	29.2	38.2	37.3	37.8	30.0	5.8	95	20	2,360
Outer forebay spill wall #18	6	28.8	37.3	38.2	37.8	30.0	6.2	100	22	2,337
Outer forebay spill wall #16	7	24.1	30.3	31.2	30.8	30.0	6.6	95	24	2,313
Outer forebay spill wall #14	8	27.3	38.0	37.5	37.8	30.0	5.2	90	24	2,343
Gatehouse exterior piers 7 & 9	9	22.9	32.2	32.5	32.4	30.0	6.5	115	23	2,280
Outer forebay spill wall #8	10	27.8	35.9	36.8	36.4	20.0	4.5	75	19	2,380
Outer forebay needle dam	11	31.4	39.5	40.5	40.0	20.0	5.2	70	13	2,363
Outer forebay spill wall #4	12	27.3	34.7	35.4	35.1	20.0	6.2	100	20	2,323
Gatehouse exterior pier #10	13	28.6	37.0	36.4	36.7	30.0	6.6	90	24	2,310
Outer forebay spill wall #3	14	28.1	37.5	36.8	37.2	20.0	4.8	70	17	2,387
Screenhouse pier #12	15	28.6	39.7	37.3	38.5	20.0	6.0	90	18	2,373
Gatehouse exterior retaining wall 11 & 12	16	24.6	33.4	34.2	33.8	20.0	5.0	90	20	2,367
Screenhouse pier #8	17	28.1	35.2	35.2	35.2	20.0	5.6	90	19	2,373
Outer forebay spill wall #2	18	32.5	41.8	42.4	42.1	30.0	5.2	75	16	2,386
Screenhouse pier #12	19	26.3	35.7	35.4	35.6	20.0	5.3	90	18	2,360
Ice diverter retaining wall	20	25.4	32.9	32.9	32.9	30.0	6.5	115	22	2,330

house and undertake repairs to the substructure. A public walkway would then be built over this structure to provide a scenic view of the Niagara River Parkway. However, demolition aggravated the substructure and complete replacement was necessary.

It was necessary to design, construct, and complete the entire walkway in less than 25 days. Three piers, post-tensioned into the bedrock, were constructed. Existing abutments were rehabilitated to accept a segmental steel truss walkway.

Schedule

The schedule only allowed for 60 days to complete the entire project. Dewatering occurred on October 6,



Fig. 4—Retaining wall construction.



Fig. 5—Deterioration of original spillway wall.

1987, and the forebays were watered up on December 7.

Although the cofferdam was designed to withstand ice loads, there was concern that the cofferdam might be difficult to remove if icing occurred. The potential cost of heating the generation station and loss of revenue if the cofferdam could not be removed more than justified the decision to schedule crews to work two 10-hour shifts every day of the week. The work force on site peaked at 160 tradesmen per two 10-hour shifts.

The main milestone activities on site were installing and removing the cofferdam, placing the gatehouse base slab, designing and building the scenic walkway piers and approval, and removing forebay debris.

Conclusion

The successful five-month work program resulted in restoring the structural integrity of the gatehouse foundation, outer forebay retaining wall, the safe operation of the gates, debris protection by the newly installed screens, and improved esthetics of the intake works. This enabled the Niagara Park Commis-

sion to realize it's long-term plans for a scenic public walkway along the Niagara River bank.

A 60-day work program was completed on schedule involving approximately 160 tradesmen working 90,000 direct and indirect man-hours. Technological advances in hydraulic equipment contributed to the timely and economical completion of this concrete restoration project.

Acknowledgments

The author wishes to thank all the associated departments within Ontario Hydro who provided design and technical advice during the construction phase, all the associated trade unions for providing qualified skilled tradesmen, and all the tradesmen for insuring the completion of the project on schedule.

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Rehabilitation of an Elevated Roadway Bridge

by Alexander M. Vaysburd

Rehabilitation of concrete in bridges prematurely damaged by deterioration continues to be a major problem throughout North America. Approximately 564,000 bridges are standing in the United States. At least 105,500 of them need repair or replacement. Millions of square feet of bridge decks show heavy deterioration—cracking, spalling, corrosion of reinforcement, etc.

One such bridge, the elevated roadway at Baltimore/Washington International Airport, was repaired between September 1988 and July 1989. Built in 1977, the bridge superstructure is supported by 152 concrete columns and two abutments on spread footings. The 1720 ft (524 m) long concrete structure (Fig. 1) consists of 73 spans in continuous units three or four spans long, with cantilevers at the end of each unit and a suspended slab from 12 ft (3.7 m) to 28.8 ft (8.8 m) long between cantilevers (Fig. 2). An expansion joint is provided at each end of the suspended spans and at each abutment.

The width of the bridge varies from 24 to 44 ft (7.3 to 13.4 m) of clear roadway with a parapet and a safety walk on one side of the bridge and a sidewalk on the other. The bridge deck is a 20 in. (51 cm) thick cast-in-place reinforced concrete slab. The positive moment regions in the deck slab in each 24 ft (7.3 m) clear roadway span contain 20 equally spaced 10 in. (25 cm) diameter voids.

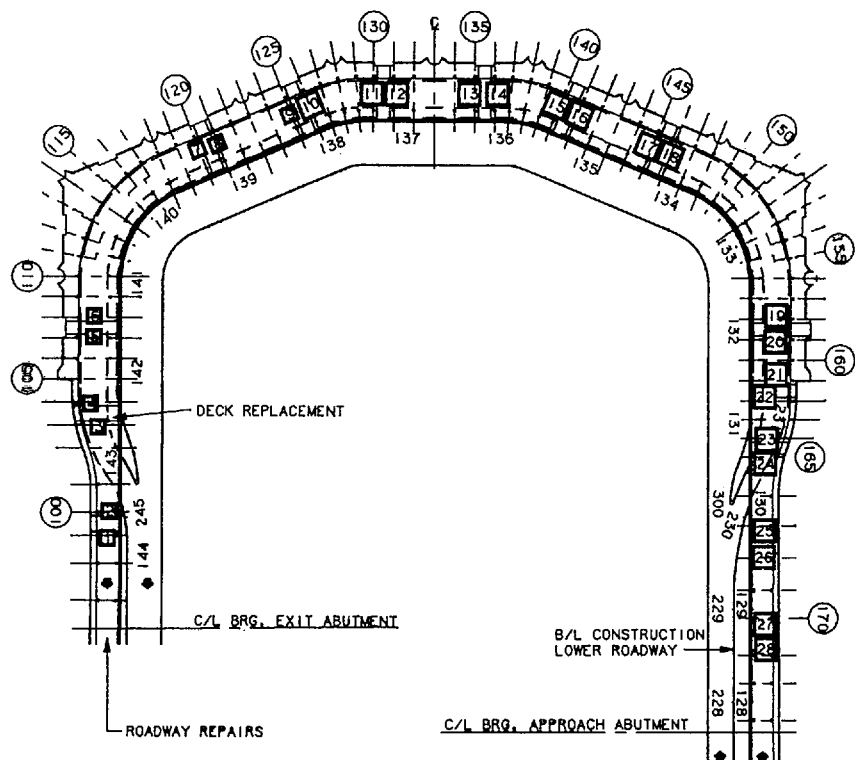


Fig. 1 — Bridge plan view.

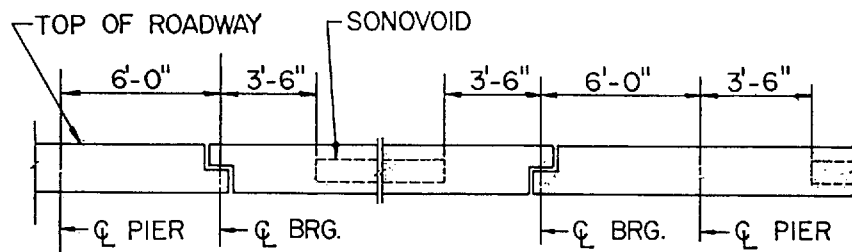


Fig. 2 — Typical suspended span longitudinal section.

The specified concrete strength for the columns and deck slab was 4500 psi (31 MPa).

Field inspection

The first phase of the project included an in-depth inspection to determine the physical condition of the structure. The bridge was visually inspected for spalls, cracks, rust stains, discoloration, corrosion of reinforcement, and other indications of concrete deterioration. The surface areas were also inspected by sounding (hammer tapping) to locate delaminated areas. The inspection yielded the following:

- The top of the slab was free of deterioration with the exception of some minor spalls along the expansion joint angles.
- Cracking, light at most locations and moderate at others, existed throughout the deck surface. Transverse cracking was predominant, although some longitudinal and random cracking was observed.
- Severe deterioration was found on the underside of the deck slab at the expansion joints. Leakage through the joints had caused extensive corrosion of the reinforcement, spalling, and delamination of the concrete cover (Fig. 3). Concrete spalls at the cantilever supports were about 4 in. (10 cm) deep at several locations and the main reinforcing bars were heavily corroded.
- The specified cover to reinforcement at the bottom of the deck slab was 1 in. (25 mm); measurements indicated that at some locations the cover did not exceed ¼ in. (6 mm).
- The concrete sidewalk at several locations had severe random cracking with crack widths varying from fine to ¼ in. (6 mm).
- Of the 30 expansion joints, 26 had preformed neoprene elastomeric compression seals. Four joints had been repaired in 1986 by replacing the existing compression seals. All of the remaining joints were showing evidence of leaks (some very severe.)

Testing program

A concrete sampling and testing program was carried out to supplement visual observations and to determine the potential durability of the concrete and the expected service life of the structure. This program included obtaining and testing cores and samples for compressive strength, chloride ion content, depth of carbonation, pH value, and for performing petrographic examinations. The sampling plan was designed to obtain samples from concrete areas judged to have light, moderate, and severe deterioration. The most severe areas of concrete deterioration were adjacent to expansion joints on the underside of the deck slab.

Cores taken near the expansion joints (Fig. 4) were visually examined and tested for compressive strength. A total of 15 cores were taken from various locations throughout the bridge.

Using the dry pulverizing method, 12 samples for chloride ion content were taken from six locations on the underside of the deck slab adjacent to expansion joints and the longitudinal construction joint. Six of these were shallow samples taken at a depth of 0 to 3 in. (0 to 7.5 cm). The remaining six were deep samples taken at a depth of approximately 5 in. (12.5 cm).

For depth of carbonation tests, cores 3 in. (7.7 cm) in diameter were obtained at three locations. Two samples of concrete dust (left over from the chloride ion testing) were taken to determine the pH value of concrete.

Compressive strength

Compression tests performed on cores taken at the top of the deck in suspended slab areas did not indicate concrete deterioration. (Compressive strength averaged 7424 psi [51 MPa].) Cores taken from the underside of the deck at the cantilever supports and adjacent areas of suspended slabs were visually examined. The top portions of the cores (inside the joint) did not show any concrete deterioration, but the

bottom portions contained deteriorated concrete from ½ in. (12.5 mm) to 3 in. (76 mm) thick. Some of the cores contained delaminations and cracks, and the reinforcement in two cores had corroded to the extent that the bond between concrete and steel was severely affected. Average compressive strength of the two cores tested was 7590 psi (52 MPa); eight cores could not be tested due to either included reinforcing bars or delamination and cracking.

Carbonation

Laboratory measurements of depth of carbonation using a phenolphthalein indicator showed that color changes occurred at an average depth of ⅝ in. (8 mm).

Tests for pH value indicated an alkalinity of the concrete of about 11.

Chloride content

Sampling and testing for total chloride content was carried out in accordance with AASHTO T-260. The shallow samples showed levels of chlorides that ranged from 0.3 to 7.4 lb/yd³ (0.18 to 4.4 kg/m³) of concrete, and the deep samples ranged from 0 to 3.2 lb/yd³ (0 to 1.9 kg/m³).

Petrographic examination

The results of a petrographic examination indicated the concrete was properly made. The entrained air-void system was not entirely adequate, but there were no signs of freeze-thaw distress. The average entrained air volume was about 5 percent. The paste was of low water-cement ratio and was hard and dense. The bond of the coarse and fine aggregates to the cement matrix was good. The paste was well cured and hydrated.

The concrete had been affected by an alkali-silica reaction that generated fine cracks in the concrete. The orientation of the cracks was both parallel and normal to the exterior concrete surface. A gross manifestation of the reaction was the map, or pattern, cracking evident on the concrete surface. The cracks were shallow at some loca-