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REHABILITATION OF BUILDINGS & BRIDGES INCLUDING INVESTIGATIONS

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GAJANAN M. SABNIS, Ph.D.P.E.

Professor of Civil Engineering
Howard University, Washington, DC

JEAN B. JOHNSON
NSF Program Manager

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TABLE OF CONTENTS

PREFACE

This volume documents the (partial) results of the International Conference on Rahabiliattion of Buildings and Bridges held in Bombay, India on December 21 - 23, 1981. It includes 35 papers either presented at the conference or prepared in conjunction with, but could not be presented due to the time constraint.

The initial concept of this conference was conceived soon after the American Concrete Institute's Chapter was established and was discussed with the local members of the chapter for co-sponsorship and implementation in 1979. The National Science Foundation was approached for travel funds for the attendees from the United States and also for distributing the proceedings of this conference in the United States. It took this organizer several trips and a great deal of efforts to motivate individuals from this country to contribute to this conference. It is noted that these efforts resulted in 32 papers from North America compared to the total of 61 papers in the conference.

The conference was a great success in that it brought together the various view-points from both the East and West in terms of methodology, techniques and individual expereinces. The papers in this volume reflect the State-of-the-Art on rehabilitation work, through codes, standards and practice, and importatntly, through the actual case studies. It is hoped that this information will be utilized by those in the field and also those interested in the topic.

The papers, which are included in this volume are only those from North America. The other papers are not included because permission could not be obtained. The volume is divided into three major parts: 1. Overview and Topics of General Interest; 2. Rehabilitation of Buildings; and, 3. Rehabilitation of Bridges.

The editor would like to thank all authors and participants who made the conference a success. The funding for these proceedings was provided by NSF Grant No. INT81-11905 from the National Science Foundation and their support is gratefully acknowledged. Drs. Jean Johnson and Osmond Shinaishin, the Program Managers from NSF were most cooperative for making the final arrangements and actual details, to whom the editor like to tahnk in a special way. Local activities and arrangements were taken care by the ACI Chapter in Bombay. Thanks are also due to them.

The volume was prepared from the camera-ready manuscripts as given by the authors. The reader is therefore urged to contact for any comments and inadvertant omissions and corrections.

Gajanan M. Sabnis
Editor

October 21, 1982
Washington, D.C.

PART I

OVERVIEW AND GENERAL TOPICS RELATED TO REHABILITATION

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REHABILITATION OF STRUCTURES: INTRODUCTION AND OVERVIEW

by

Gajanan M. Sabnis
Professor of Civil Engineering
Howard University, Washington, D.C.

SUMMARY

This paper presents an overview of rehabilitation of structures. The overview is based on the ideas and views of the author with his experience in the United States; however, it is also applicable to other countries. The main topics addressed are: (a) reasons, (b) problems with materials, stresses and loads, (c) techniques, (d) codes and guidelines, and (e) future potentials for this type of work.

INTRODUCTION

There has been a growing concern both in the developed and developing countries concerning the rehabilitation problem in structures. The structures include both the buildings and bridges. Man needs to live in a shelter and needs to move around on roads of which the bridges are the integral part. Thus, the need to accomodate a growing population and the concern for the old buildings "for their aging and deterioration" have drawn the attention of engineering professionals and governments at the local, state and federal levels. In the case of bridges, the need is to accommodate not only the growth in traffic for its intensity, but also their "natural deterioration and inadequacy".

One of the approaches taken in the past has been to check on a case to case basis if the structure would adequately survive the needs with the change of time. If it did, then depending on the extent, the structure was either used with "band-aid" operation or was torn down to make room for a structure much stronger using modern materials. Obviously, these approaches have been successful in isolated cases and their generalization is not an easy extrapolation. The costs of replacement run into billions of dollars and, hence, the approach recently has been to take a fresh look at the overall problem and arrive at "Rehabilitaion Solutions in a Cost Effective Manner".

In order to tackle this problem on an overview basis, the following topics are presented in this paper:

a. Reasons for Rehabilitation;

b. Problems Associated with Rehabilitation:

   (1) Materials

   (2) Stresses

   (3) Loads

c. Codes and Guidelines for Rehabilitation;

d. Techniques; and

e. Future Potentials - Challenges.

More emphasis in this paper is on buildings. The topic of bridge rehabilitation is treated in another paper in the conference.

REASONS FOR REHABILITATION

Reasons for rehabilitation are numerous. There is always a continuous interest in society in "Old" or "Historic" structures. Such renovation has traditionally formed a special field and needs special skills to not only preserve the old structures from a structural point of view, but also from a non-structural view point. Along with the structure, the contents are also important. The spirit of preservation and recycling of old buildings has been characterized as "creative adaptation, pride in our heritage, a link with the past, respect for the aesthetics and craftmanship of another time, insights into our development, ample creative opportunity for architectural innovation and problem-solving, enhancement of the urban fabric, greater security, stability and beauty, while conserving basic materials and meeting modern needs. The new look at rehabilitation, in the present context, is in terms of the need for additional living space and the availability of good structures which can be used for another purpose at the present time. The factors that can contribute to the rehabilitation of old buildings are as follows:

a. Need for more space;

b. Availability of old buildings;

c. Cost-effectiveness; and

d. Government incentive for such work.

In a densely populated urban environment we must look more closely at the existing space and see how it meets the new needs. The old buildings frequently can meet these needs at a reasonable cost. They have been conservatively designed for their time and soundly constructed. Thus need and the availability are necessary conditions for rehabilitation work, but they alone are not sufficient to generate the economic forces for its implementation. The economic incentive, therefore, plays an important role in rehabilitation. If economic gain is made possible by depreciation write-off tax advantages, then obviously it draws the attention of private financing sources for investments. In the U.S.A. these have been enhanced by the continuous tax reforms, when Congress has extended tax incentives to the preservation of historic buildings and rehabilitation of old buildings, generally. Often "old" buildings which have not been rehabilitated for at least twenty years qualify for a write-off on rehabilitation expense.

The problem of successful renovation is the adaptation of the old space for the new use of that old space with safety and at reasonable cost. The space modifications may be relatively minor, with the addition or removal of only a few partitions, or it may be major, with the removal of the interior and renovating it with new elements needed for proper uses of the structure. The exterior walls may undergo only minor modifications, per-

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haps only cosmetic; in some cases by strengthening they often can be saved, but in some masonry buildings with wood floors, the latter may have to be removed and replaced by metal or concrete floor systems. In some cases the addition of stairs or elevators may be required. An assessment of fire safety may lead to a variety of required modifications: installation of fire detection and/or supression system, addition of fire-proofing, fire separation walls, and fire doors and other exits. Trade-offs between alternative solutions would determine the final architectural space modifications.

PROBLEM AREAS

Problems associated with rehabilitation work arise from the following main reasons:

a. Materials;

b. Stresses; and

c. Loads.

A majority of old buildings which could be rehabilitated were used originally for a purpose other than their future use. In assessing the adaptability to new uses it is important to look into the above three aspects. All of these can be put together as the basic structural design criteria. Design live loads for different types of buildings recommended by different cities in 1900 with typical live loads specified in contemporary design are shown in Table 1 based on Bresler's work (3). It is obvious that new buildings are designed for about one-half thelive loads used in the design of old buildings.

LIVE LOADS FOR FLOORS FROM VARIOUS BUILDING (Ref. 3)

Occupancy	New York 1900	Chicago 1898	Philadelphia 1899	Boston 1899	UBC 1980
Apartments Hotels	60*	40	70	50	40
Office Buildings	75	100	100	100	50
Schools	75	--	--	80	40
Public Assembly	90	100	120	150	50-125
Light Industry	120	100	120	--	75
Heavy Industry	150	--	150	250	125

* All values are in psf.

TABLE 1

Similar comparison of recommended working stresses for different construction materials with those generally used at the present time is shown in Table 2. The allowable stresses for rolled steel shapes used were about 2/3 of the present values. Likewise, the allowable shear and bearing stresses for these connections were as low as 40% for shear and 30% for bearing. Developments in changes of allowable stresses for concrete during the last 75 years of ACI have taken place, particularly in the shear stress in reinforced stresses concrete. These changes for concretes of different strength are shown in Table 3.

ALLOWABLE STRESSES FOR CONCRETE AND STEEL (psi.) (Ref. 3)

Materials	New York 1900	Chicago 1898	Philadelphia 1899	Boston 1899	UBC 1973
ROLLED STEEL					
Tension	16,000	15,000	14,500	15,000	22,000
Compression ($\frac{\ell}{r} = 0$)	15,200	15,000	14,500	12,000	22,000
Bending	16,000	16,000	--	16,000	24,000
Shear	9,000	--	8,750	10,000	14,000
CONCRETE					
Compression	230	55	210	--	500*
Bending	30	--	--	--	1,030+

$*f_c = 0.2f'_c = 500$ psi; $+ f_b = 0.45f'_c = 1{,}030$ psi.

TABLE 2

ALLOWABLE SHEAR STRESS FOR REINFORCED CONCRETE BEAMS IN ACI HISTORY (Ref. 3)

CODE	YEAR	Concrete Cylinder Strength f', psi		
		2000	3000	5000
J.C. Progr. Rep.	1909	105	105	---
NACU (ACI),	1910	175	175	---
1st Joint Comm.,	1916	105	160	---
ACI	1917	130	200	---
ACI	1920	105, 210*	160, 320*	---
2nd Joint Comm.,	1924	105, 210	160, 320	---
3rd Joint Comm.,**	1940	105, 210	160, 320	260, 520
ACI	1951	140, 210	210, 320	280,420
ACI	1956	140, 210	210, 320	210, 320
ACI (App. B)	1977	245	300	350

All values calculated from v/bd and adjusted for j = 0.875

TABLE 3

*Lower limit value for beam reinforcement without special anchorage; higher limit value for beam reinforcement with special anchorage.

**For values of shear stress greater than the lower limit... all shear must be taken by web reinforcement.

Review of the details on material properties, applied loads and allowable stresses presented here indicates the complexity involved in the renovation work. One must survey not only the conditions as they exist at the time of rehabilitation but also research "the period" when the original building was constructed to be aware of complete range of materials, the then "design" loads and the allowable stresses.

If the drawings and specifications for the original building are obtained, the engineer's problem is greatly simplified. Therefore, it is important to devote some efforts in locating such documentation. Often the old architectural and engineering firms pass on their archival materials to their former associates, or to libraries, museums, and educational institutions. In lieu of such information one has to rely on information contained in old texts, manuals, technical publications, and his prior experience and judgment.

CODES AND GUIDELINES FOR REHABILITATION

In the United States, the task of rehabilitation is considered quite seriously and has resulted into some code provisions and guidelines. The Rehabilitation guidelines (4) were prepared for the Department of Housing and Urban Development (HUD) by an act of U.S. Congress in 1978. They are not a code; rather they are designed for voluntary adoption and use by the states and local communities as a means to upgrade and preserve the nations stock while maintaining reasonable standards for health and safety. The term "rehabilitation" as used in the guidelines, includes any set of activities related to the general view of existing buildings as a resource to be conserved, rehabilitated, or reused.

The present edition of the Rehabilitation Guidelines is published in eight separate volumes. The first four guidelines are designed for use by building officials, members of the executive and legislative branches of the government, and related commissions and organizations involved in developing or implementing building regulations. These guidelines cover the following topics:

1. The Guideline for Setting and Adopting Standards for Building Rehabilitation provides an introduction and background to the building regulations that affect rehabilitation. It describes methods for identifying regulatory problems in a community, and recommends ways to amend, modify, or supplement existing regulations to encourage rehabilitation.

2. The Guidelines for Municipal Approval of Building Rehabilitation examines the inherent differences between regulating new construction and regulating rehabilitation, and presents specific recommendations for dealing with rehabilitation within municipal building departments.

3. The Statutory Guideline for Building Rehabilitation contains enabling legislation that can be directly adopted by communities to provide the legal basis for promoting rehabilitation through more effective regulation.

4. The Guideline for Managing Official Liability Associated with Building Rehabilitation addresses the liability of code officials involved with the administration and enforcement of rehabilitation, and provides recommendations for minimizing liability problems.

The remaining four guidelines are technical in nature, and are intended for use by code officials, inspectors, designers, and builders. They cover the following topics.

5. The Egress Guideline for Residential Rehabilitation lists design alternatives for the components of egress that are regulated by current codes such as number and arrangement of exits, corridors, and stairs, travel distance, dead-end travel, and exit capacity and width.

6. The Electrical Guideline for Residential Rehabilitation outlines procedures for conducting inspections of electrical systems in existing buildings, and presents solutions to common problems associated with electrical rehabilitation such as eliminating hazardous conditions.

7. The Plumbing DWV Guideline for Residential Rehabilitation presents criteria and methods for inspecting and testing existing drain, waste, and vent (DWV) systems, relocating fixtures, adding new fixtures to existing DMV systems, extending existing DWV systems, and installing new DWV systems in existing buildings.

8. The Guideline on Fire Ratings of Archaic Materials and Assemblies contains the fire ratings of building materials and assemblies that are no longer listed in current building codes or related reference standards. Introductory material discusses flame spread, the effects of penetrations, and methods for determining the ratings of assemblies not listed in the guideline.

A specific section related to the format of these guidelines which can be adopted at the local level is presented in APPENDIX 1.

The ATC-3: Provisions for the Development of Seismic Regulations for Buildings (5). Contains recommendations in earthquake prone areas. The seismic rehabilitation is considered in the paper by Iyer and Aroni (6).

TYPES OF REPAIR & STRENGTHENING

Strengthening a building for adequate seismic response is a unique problem in design, and each building poses special constraints for design of appropriate strengthening at a reasonable cost.

Two basic types of strengthening should be considered:

(a) Maintaining Original Structural System and Strengthening individual members or connections. In correcting deficiencies in members, it is vital not only to ascertain the mechanical properties of the material, but also to determine geometrical properties in order to permit a necessary review of design calculations.

The manner in which a member is loaded under service conditions may dictate the means for increasing its capacity. In the case of compression members, reducing unsupported length, increasing the cross-sectional area, or replacing sections with higher strength material may be used. Tension members, on the other hand, are usually strengthened by providing additional section area, or by replacing them with the same size sections of higher-strength material. Enhancing the capacity of flexural members usually depends on the variation and type of loading which greatly influence the selection of remedial measures.

(b) Modification of Structural System: A variety of factors, including structural, architectural, economic, or societal needs, may preclude preservation of the original structural system with appropriate strengthening. In such cases substantial modifications of the structural system may be necessary to accommodate changes in the use of the building, new loading patterns, or specified increased level of any other resistance.

The process of integrating an original structural system with new structural components, such as additional members, frames, or walls, and replacement of floor or roof systems, is similar to that entailed in developing a new design. An analysis of the modified system will require investigation of stresses and deformations under gravity load as well as under combined effects of gravity and lateral forces. Any deficiency in the original components of the system would require strengthening with due regard to all the factors described in the preceding section.

A variety of modifications may be introduced. Wood floor systems may be strengthened or replaced by steel decking or reinforced concrete slabs. Lateral bracing may be strengthened or replaced by reinforced masonry or concrete shear walls. In such cases connections of the new walls to the floor must provide for transfer of the design lateral loads.

Introduction of new elements or stiffening of existing elements to resist lateral forces may significantly alter the earthquake response of the resulting structure. Larger lateral forces may be induced in stiffer buildings and these should be taken into account in the modification design. Whenever possible, assymmetry in adding new components should be avoided because of torsional effects and possible increased lateral forces in critical components.

TECHNIQUES OF REPAIR AND STRENGTHENING

Various aspects of repair and strengthening such as (a) available materials, (b) repair techniques, and (c) strengthening.

(a) AVAILABLE MATERIALS

The more common materials used to repair or strengthening monolithic reinforced concrete construction are as follows:

1. Shotcrete: Shotcrete, known also as "gunite", is pneumatically applied concrete. It may be applied by a "wet mix" or "dry mix" process. Wet mix involves pumping of premixed cement mortar to a nozzle where compressed air impels it onto the substrate surface. To ensure a plastic mix either higher water content or special plasticizers are used in the wet mix. In some cases there may be difficulties in obtaining proper embedment of steel bars when using shotcrete. Where high water content mortar is used the shrinkage may also be high. Dry mix involves transporting premixed cement and sand by compressed air to the end of the hose where water is injected and mixed with the dry blend and the mortar is impelled against the substrate surface. While lower water content can often result in high-strength and low-shrinkage material, the quality of the work generally depends on the skill and workmanship of the men controlling the nozzle and the flow of water to it. The general practice of shotcrete application is described in the ACI Standard ACI 506-66 (reaffirmed in 1972), "Recommended Practice for Shotcrete".

2. Epoxy Resin: Epoxy resin is a general classification of adhesives manufactured from petroleum products and usually consists of two or more component chemicals mixed immediately prior to application. The various chemical compositions of these agents continually change to meet new performance requirements. For structural purposes, the material selected should contain 100 percent reactive solids. Detailed descriptions of epoxy resins and recommended uses are provided in manufacturers' catalogs and other publications. A procedure for verifying specifications and quality control should be established for selected materials. The viscosity, setting time, curing conditions, and mechanical properties of a mixture depend on the components of the mixture. The most common of these resins used in repairing building cracks are of the low-viscosity type, meaning that they can be mixed and pressure-injected into very small cracks. In order to fill spaces between cracked surfaces before the mixture has set, it is necessary that the epoxy not harden too quickly. Upon hardening, these materials strongly adhere to adjacent concrete and steel surfaces.

   Higher-viscosity epoxy resin mixtures can be used for surface coating or for filling larger cracks or holes. Epoxy-resin mixtures are higly toxic and the chemical reaction resulting from mixing the components is exothermal. The heat generated by the reaction can cause the mixture to boil if too large a volume of material is confined with no outlet for heat dissipation. Epoxy mixture strength characteristics should be provided by manufacturers. Some components of epoxies deteriorate with time in storage, and some verification of epoxy properties used in the field may be advisable.

3. Epoxy-Mortar: For larger voids, it is possible to combine either the low or high-viscosity epoxy resins with sand aggregates providing a heat sink and increasing the modulus of elasticity. Epoxy-mortar mixtures have higher tensile strength, higher compressive strength, and greater shear capacity, but a low modulus of elasticity, than Portland cement concrete. Thus, epoxy mortar is not a compatible-stiffness replacement material for reinforced concrete. Changes in the mechanical properties of epoxy mortar with large variations in temperatures must be considered when a large volume of replacement material is used.

Epoxy resin and mortar are combustible, and for most epoxies strength and stiffness decrease approximately linearly from values at about 70^{o} F to zero at about 400^{o} F. Therefore, to maintain required fire endurance, epoxy-repaired elements must be fireproofed when repair and strengthening are required. The probability of a severe earthquake occurring shortly after a severe fire is rather small, but the probability may be of greater importance when an earthquake is followed by a fire in a building previously repaired with epoxy materials.

A discussion concerning the applications of epoxy compounds in repair of concrete can be found in the ACI Report of Committee 503 entitled, "Use of Epoxy Compounds with Concrete".

4. Fiber-Reinforced Concrete: A material that is stronger in tension than the original material can be obtained by adding steel, glass, or plastic fibers to normal or Type III Portland cement concrete. The fire resistance of the selected fiber concrete must be determined before use in repair and strengthening when required for gravity load conditions. A discussion concerning the applications of fiber reinforced concrete can be found in the ACI Report of Committee 544, entitled, "State-of-the-Art report on Fiber-Reinforced Concrete".

5. Gypsum Cement Concrete: Gypsum cement concrete has not been widely used in structures. Typical properties of structural gypsum cement mortar are given in Table 4. Of the three materials listed, gypsum cement mortar has the lowest tensile strength, but its modulus of elasticity is close to that of concrete.

6. Portland Cement Concrete: Type III cement for mixing high early strength concrete has been used for many years. The properties of the concrete are described in appropriate ASTM specifications.

7. Quick-Setting Cement Mortar: Quick-setting cement mortar is a relatively new material patented by Republic Steel Corporation and originally developed for repairing reinforced concrete floors adjacent to steel blast furnaces. It is a nonhydrous, phosphate-magnesium cement with two components, a liquid and a dry aggregate, mixed similarly to Portland cement concrete. The cement mortar must be placed and cured in a water-free environment. Its properties are summarized in Table 4.

8. Preplaced Aggregate Concrete: This material, also known as "intrusion" or "grouted concrete", derives its name from the method of construction in which forms are first filled with clean, well graded coarse aggregate and then mortar or grout is pumped into the void spaces. Intrusion mortars usually include expansive admixtures to ensure proper bond between aggregate and mortar. This material is well suited for restoration work, especially where access is difficult or congestion of reinforcement makes shotcreting difficult.

9. Reinforcing Steel: Studies on anchorage of dowels and reinforcing bars in existing concrete have been limited. A common technique for providing anchorage is as follows: A hole larger than the bar is drilled and filled with epoxy, expansive cement, grout, sulfur, or other high-strength grouting material. The bar is then pushed into place and held until the grout is cured. Field tests have shown that when the embedment length recommended in the ACI code is used the ultimate strength of the dowel or bar can be developed. In

TABLE 4

TYPICAL STRENGTH AND STIFFNESS CHARACTERISTICS

OF SOME REPAIR MATERIALS (psi)

Material	3-Day Compressive Strength	28-Day Compressive Strength	3-Day Tensile Strength	28-Day Tensile Strength	2-Day Modulus of Elasticity	28-Day Modulus of Elasticity
Epoxy - Neat	10,400	12,000		4,900*	260,000	490,000
Epoxy - Mortar	8,400	9,900	740	920	$2X10^6$	$2X10^6$
Gypsum Cement Mortar	4,800	7,200	430	570	$3.2X10^6$	$4.2X10^6$
Quick Setting Cement Concrete Mortar	5,000	7,900	420	610	$3.6X10^6$	$4.2X10^6$

*Based on 14 days.

order to determine the minimum embedment length required to develop the tensile strength of bars which can be used with epoxy or high-strength grouting materials.

10. Mechanical Anchors: Mechanical anchors use wedging action to provide anchorage. Some anchors resist both shear and tension, while others resist only tension. The manufacturers of these mechanical connectors have specific recommendations for installation and the strengths that will develop. Most of these strengths were obtained by static tests on selected materials. A limited number of dynamic tests have been conducted for specific applications. Anchor strength should be determined by an independent laboratory for specific applications, or code values with a large factor of safety should be used.

(b) REPAIR

Repair techniques must be selected according to the degree of damage and the level of repair to be accomplished. Two types of repair must be distinguished: (1) where damage is limited to moderate cracking, and sound reinforced concrete can be restored by filling in the cracks, and (2) where damage involves extensive cracking and spalling, and some of the concrete is shattered, but sound reinforced concrete can be restored by shotcreting or other means of replacing damaged concrete bonded to reinforcing steel.

1. Small Cracks: If concrete cracks are reasonably small (opening widths of less than 1/4 inch), the simplest method of repairing reinforced concrete elements is to pressure-inject epoxy. The procedure for epoxy injection is described below.

   External surfaces are first cleaned of nonstructural materials. Plastic injection ports are placed along the surface of the crack on both sides of the member and secured in place with an epoxy sealant. The center-to-center spacing of these ports should be between 1 and 1.2 times the thickness of the concrete element. However, the spacing is dependent on the width of the element and whether or not pressure injection will be accomplished from both or only one side of a member. After these ports are in place, the surface of the crack between ports is sealed with epoxy sealant.

   After the sealant has cured, a low-viscosity epoxy resin is injected into one port at a time, beginning at the lowest point of a vertical crack or at one end of a horizontal crack. Working at a port, the epoxy is pressure-injected until the material flows on the same side of the member. When flow is observed, the injection port is closed and all ports have been used, the final port is closed until the epoxy is cured. This is normally a two-man operation, with epoxy injections occurring from one side.

   Smaller cracks require higher pressure or more closely spaced ports to penetrate the depth and width of the member with epoxy. Larger cracks allow larger port spacing, dependent on the width of the member. Epoxy injection is appropriate for all types of structural elements... beams, columns, walls, and floor units. Members repaired by this technique and subject to loading conditions similar to those having caused damage have shown failure cracks adjacent to epoxy repairs; e.g., the repair is stronger than the adjacent concrete material. However, the failure mechanism for the structure is not altered by the repair.

   If there is a loss of bond between the reinforcing bar and concrete through a number of cycles of deformation, concrete adjacent to the bar is pulverized to a very fine powder and effectively creates a dam and prevents the epoxy from saturating the region. Pressure injection of cracks cannot restore bond. Also, cracks smaller than 0.003 inch may be difficult to pressure-inject effectively. Unrepaired small cracks and loss of bond result in a structural system less stiff than the original.

   With full penetration of epoxy, original strength can be restored. However, recovery of only 70 to 80 percent of the original strength should be assumed. Because the member has been damaged, it is probable that the original section may not provide sufficient strength for the structure. Therefore, a technique for strengthening elements to avoid similar damage during subsequent earthquakes should be considered.

2. Large Cracks and Crushed Concrete: For cracks larger than 1/4 inch or for regions in which concrete is crushed, treatment other than by epoxy injection is required. Loose concrete should be removed, leaving only solid material. Material that is removed can be replaced with shotcrete, any one of special cement mortars, or concrete replacement. Selection of replacement material depends on the desired material characteristics as described earlier in this Section. When damage is severe, the need for additional shear or flexural reinforcement should be considered. If, however, development of the additional reinforcement into adjacent solid concrete regions is required, the repaired section will be stronger than adjacent existing material and failure in this adjacent section would be probable during a subsequent earthquake.

   In the case of damage to wall and floor diaphragms it may be more economical to add new steel on the outside of surfaces and to cover this steel with concrete rather than to repair the damaged material. The increased weight from the materials must be considered in re-analyzing building forces and in checking the foundations.

3. Reinforcement: In a severely damaged reinforced concrete member reinforcement must have buckled, elongated with excessive yielding, or fractured in extreme cases. Reinforcement can be replaced with new steel, using butt welding, lap welding,

or in some cases by a splice. If practical, the repair should be made without removing the existing steel. The best approach depends on the amount of space available in the original member. Additional confinement steel should be added to inhibit future buckling of bars in this region. The additional steel will not substantially increase the strength of the member, but will extend its inelastic strength carrying capacity.

(c) STRENGTHENING

If the decision to strengthen or stiffen a building is made during the process of repair, a thorough analysis of the structural framing system must be made. The level to which a system should be strengthened or stiffened must be determined in accordance with the established criteria.

1. Replacement of Structural System: Loading conditions must be evaluated at the time of strengthening and a new structural system selected as for a new building. However, particular care must be exercised in tying horizontal floor diaphragms into the lateral force-resisting system. Although the lateral-vertical load-carrying system has increased strength, if the connections to horizontal diaphragms are inadequate, this junction between the stiffened system and existing floor diaphragm will subsequently fail. The techniques discussed above for repairing and adding new materials to damaged regions are applicable. Foundations must be reviewed on the basis of the increased weight of the structural system.

2. Addition of New System to an Existing Structure: If new structural systems are added to the existing system, the existing internal stresses in members must be considered in analyzing behavior during subsequent overloads caused by earthquakes. Appropriate strengthening can delay failure or prevent structural damage from an earthquake. A smooth transition of stiffness and strength in the structural system must be provided similar to that for new construction. Increases in column and girder sizes can be accomplished by adding reinforcement adjacent to existing columns, by adequately confining steel, and by providing concrete cover for the additional width and depth of the member. Existing shear walls can be strengthened by providing additional reinforcing steel on the outside of the walls and increasing wall thickness with additional concrete. Care must be taken to anchor the ends of horizontal and vertical reinforcement into adjacent columns and beams in order to provide an integral wall unit.

CONCLUDING REMARKS

The problem of rehabilitating old structures presents an inherent challenge to the structural engineer. It demands of him the knowledge of the history (to appreciate the importance of the problem), the art of construction that the forefathers practiced. Combined with these, he should know recent scientific tools, such as gamma ray scanning devices, sonicscopes, and pachometers which are of particular help for looking into the concrete and steel of the structure. Finally, experience, judgment, and imaginative ideas of structural action should enable him to integrate the new technology materials and structural concepts with the old building. Primary responsibility for all these works, should be safety of the "rehabilitated" structure for its new function. The other papers of this symposium deal with detailed study cases and experiences, and the proceeding volumes as a whole should become a valuable addition to the library of those interested in this topic.

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APPENDIX 1*

FORMAT AND METHODOLOGY FOR DEVELOPING A LOCAL REHABILITATION CODE, REGULATIONS, OR GUIDELINES

The approach presented here deals with the safety and health objectives of building regulation, since it is assumed that accepting reduced levels of performance related to these two areas will be more difficult to justify than similar reductions related to welfare or to property protection. However, similar approaches can be developed for analysis related to these goals also.

*Reproduced from "Rehabilitation Guidelines 1980", Published by the U.S. Department of Housing and Urban Development, 8 volumes, available from the Superintendent of Documents, U.S. Government, Printing Press, Washington, D.C.

When considering the rehabilitation of a given existing building, it is necessary to analyze its intended use and occupancy, in order to determine what levels of performance should be required by the regulation. It is useful to consider separately three categories of attributes for which performance is regulated by codes:

(a) Structural safety;

(b) Fire safety; and

(c) Accident safety, health and hygiene.

In the suggested analytical approach, the proposed use of the building is analyzed by considering a series of matrices. Each matrix requires the consideration of a set of code regulated attributes with respect to each occupancy group. For purposes of illustration, the CABO/BCMC occupancy index is used. However, a community applying this approach should substitute the occupancy classifications in its building code. The occupancy designations and brief descriptions are as follows:

Group A - Assembly occupancy is the use of a building or structure, or any portion thereof, for the gathering together of persons for purposes such as civic, social or religious functions or for recreation, or for food or drink consumption or awaiting transportation.

Group B - Business Occupancy is the use of a building or structure, or any portion thereof, for office, professional, or service type transactions including normal accessory storage and the keeping of records and accounts.

Group E - Educational Occupancy is the use of a building or structure, or any portion thereof, for the gathering together of persons for the purpose of instruction.

Group H - Hazardous Occupancy is the principal use of a building or structure, or any portion thereof, that involves highly combustible materials or flammable materials, or explosive materials that have inherent characteristics that constitute a higher fire hazard.

Group F - Factory-Industrial Occupancy is use of a building or structure, or any portion thereof, for assembling, disassembling, repairing, fabricating, finishing, manufacturing, packaging or processing operations that are not otherwise classified in this code.

Group I - Institutional Occupancy is use of a building or structure, or any portion thereof, for the purpose of providing medical treatment or care and sleeping facilities of persons who are mostly incapable of self-preservation because of age, physical or mental disability, or because of security measures not under the occupants' control.

Group M - Mercantile Occupancy is the use of a building or structure, or any portion thereof, for the display and sale of merchandise.

Group R - Residential Occupancy is the use of a building or structure, or any portion thereof, for sleeping accomodations and is not classed as an Institutional Occupancy.

Group S - Storage Occupancy is the principal use of a building or structure, or any portion thereof, for storage that is not classed as a Hazardous Occupancy or for the purpose of sheltering animals.

A more detailed occupancy description is included in the model codes.

(a) Structural Safety

A community might find it useful to carry out the analysis by considering in detail a matrix which addresses each of the building code's occupancy groups, and three attributes of structural safety.

	Vertical Live & Dead Loads	Seismic Loads	Wind Loads
A - Assembly			
B - Business			
E - Educational			
F - Factory			
I - Institutional			
M - Mercantile			
R - Residential			
S - Storage			

1. Vertical Live and Dead Loads

It is unlikely that lower levels of performance may be acceptable here. If the proposed occupance results in increased vertical live or dead loading, the building must be capable of supporting this loading utilizing design stresses permitted in the current building code. Where the building is constructed with archaic materials, appropriate research sources data should be used in context with the historical experience of that type of construction material. Factors of safety required for archaic materials should be comparable to those required by the current building code. In the event it is not possible to establish allowable design stresses for a design analysis, it may be necessary that load tests be conducted. The amount of load to be applied varies depending upon the materials utilized since all materials do not perform in an identical manner. Higher factors of safety are sometimes applied to concrete, masonry and wood construction than are applied to other materials due to their inherent natural variability. Most codes and a variety of national standards prescribe load test procedures.

2. Seismic Loads

Buildings built to comply with earlier editions of building codes are likely not to have been designed for the magnitude of seismic forces required by the current building code.

Appendices 9 and 10 contain two specific examples of regulations establishing reduced requirements for seismic design in rehabilitated buildings when compared to new construction requirements. A community must carefully analyze its building stock in relation to its seismic risk.

It must determine for the proposed occupancy results in an increase to "life risk", from a structural viewpoint, in the event of building failure. As a general premise, one could assume that if the proposed use contains a greater number of occupants it would increase the "life risk" in the event a building collapsed during an earthquake. A further consideration would be the number of hours a day or days per week that the building is occupied, considering the probability of an earthquake occurring when the building is occupied. Also, such an analysis may consider the relative importance of particular buildings or classes of buildings to the community (e.g., hospitals, power stations, fire stations, etc.).

Reference 5 contains a chapter on Systematic Abatement of Seismic Hazards in Existing Buildings. Following procedures of this type may be desirable for buildings being rehabilitated or undergoing occupance changes in high seismic risk areas.

3. Wind Loads

Wind forces must be considered as well seismic forces when buildings undergo rehabilitation or a change in occupancy or use. Seismic forces may be more critical, however, since earthquakes cannot be predicted and occupants are unable to evacuate the structure when an earthquake occurs. Occupants of structures located in areas subject to strong wind forces such as tornados or hurricanes are generally warned well in advance of the event and can go to areas of refuge.

Additionally, if a building has been in existence for a number of years, it has probably been subjected to the maximum expected wind force for the area, except in specific hurricane areas. Accordingly, one could reasonably assume that wind design would not be a major consideration for buildings undergoing rehabilitation or a change of occupancy, and a reduction of the level of performance required for building rehabilitation compared to that for new construction, for most occupancies, may be more acceptable and easier to justify.

(b) Fire Safety

Codes provide for life safety in buildings by regulating various fire safety features associated with the buildings' intended use. The basic premise is to assure that all occupants in all occupancies are provided with an equivalent level of life safety. These regulations are based on various considerations including ignition hazards, fuel loading, occupant density, panic, sleeping, etc. For new construction these regulations are set down in a straight forward manner in all building codes. However, when an existing building is being rehabilitated or changed from one occupancy to another, the issue is more complex.

If a community wishes to explore the possibility of modifying or waiving new construction fire safety requirements for buildings being rehabilitated or undergoing a change of occupancy, while maintaining a reduced but acceptable level of safety, it must evaluate the fire safety features of existing buildings relative to the hazard of the proposed new occupancy. In some cases, the interaction of fire safety features and hazards between existing buildings and proposed use will be acceptable. In other cases, the interaction may even make the building unsuitable for conversion to the new use. A methodology should be developed for analyzing particular existing buildings for specific proposed occupancies. Such a methodology may consider various fire-related hazards, such as:

1. Ignition Hazards: The hazard due to open flame, heating, cooking or electrical equipment.

2. Smoldering Fires: The hazard of fire developing undetected.

3. Spread of Fire: The hazard of fire spreading in the building once ignited. This is controlled by limitations on flame spread on finished materials, especially in exitways and corridors. Also by amount of combustibles in the building assembly.

4. Spread of Smoke: Smoke is the primary life hazard. It spreads through unenclosed stairways and vertical shafts, open doors, ducts, etc. It may cause panic.

5. Panic: The hazard relates to building occupants' behavior, and partly depends on familiarity with the building, number of occupants, etc.

6. Exiting: The means of exiting from or within the building to a place of refuge within a given time period. Hazard is controlled by limitations on deadend corridors, enclosure of stairways, doors and closures, and similar means.

7. Community Safety: The hazard of fire spreading to adjacent buildings. Prevention of fire spread between any two buildings is dependent on the buildings' spatial relationship, type of construction, roof covering, wall protection and reasonable expectations of the capability of the fire suppression services.

A matrix relating such hazards to occupancy groups may be useful in the analysis.

PROPOSED OCCUPANCY	HAZARD (1) Ignition	(2) Smoldering fires	(3) Spread of fire	(4) Spread of smoke	(5) Panic	(6) Exiting	(7) Community safety
A - Assembly							
B - Business							
E - Educational							
F - Factory							
I - Institutional							
M - Merchantile							
R - Residential							
S - Storage							

A community may rank order the hazards for each occupancy category based on knowledge of the building stock, fire history and local firefighting capabilities. Such a rank ordering, reflecting specific community characteristics, case of rehabilitation or change of occupancy, without incurring an unacceptable level of risk.

(c) Accident safety, Health and Hygiene

Accident safety, health and hygiene are each regulated by a variety of building code provisions. Current regulations require that buildings should be brought to a condition of safety commensurate with that required for new buildings, when undergoing extensive rehabilitation or change of occupancy. A community may analyze its particular situation to determine specific areas where less than full compliance with new construction requirements would be acceptable in rehabilitated buildings, without incurring an unacceptable level of safety.

ENVIRONMENTAL REQUIREMENTS	Assembly	Business	Educational	Factory/Industrial	High Hazard	Institutional	Mercantile	Residential, Multifamily	Residential, 1 & 2 Family	Storage
o PREMISES CONDITION										
Rubbish, weeds										
Grading and drainage, ponding										
Insect and rodent control										
Paved areas repair										
Exhaust vent discharge										
o EXTERIOR STRUCTURE										
Weatherproof roof										
Weatherproof walls										
Weathertight and operable openings (doors and windows)										
Glazing										
o INTERIOR STRUCTURE										
Lead-based paint										
Bathroom and kitchen floors										
Treads and risers - uniform dimensions										
Obstruction in egress - headroom and width										
Handrails, guardrails										
Walls and ceilings - structurally stable										
Floor surfaces - uneven, obstacles										

LIGHT, VENTILATION, AND SPACE REQUIREMENTS

	Assembly	Business	Educational	Factory/Industrial	High Hazard	Institutional	Mercantile	Residential, Multifamily	Residential, 1 & 2 Family	Storage
o LIGHT										
Window area, no artificial										
Halls and stairways										
Other spaces										
o VENTILATION										
Window area										
Toilet rooms, window or mechanical										
Cooking facilities										
Mechanical ventilation, if not natural										
o DWELLING UNITS										
Privacy										
Common access										
Basement rooms										
o SPACE										
Floor area										
Ceiling heights										
Acoustics										

PLUMBING FACILITIES

	Assembly	Business	Educational	Factory/Industrial	High Hazard	Institutional	Mercantile	Residential, Multifamily	Residential, 1 & 2 Family	Storage
o REQUIRED FACILITIES										
Dwelling unit water closet & lav										
Accessibility, adequate fixtures										
o TOILET ROOMS										
Privacy										
Accessibility										
Floor, partition, wall materials										
o FIXTURES										
Quality										
Connections										
Operation and functional adequacy										
o WATER SUPPLY										
Hot and cold										
Contamination										
Water heater										
o SEWAGE SYSTEM										
Sewer functionally adequate										

TOTAL CONCEPT FOR CONCRETE REHABILITATION

BY

Floyd Dimmick, Sr.

President

Thermal-Chem, Inc.

Elk Grove Village, Illinois U.S.A.

SUMMARY

This paper describes different types of concrete deterioration and proven epoxy repair techniques used on concrete structures below and above the waterline for structural rehabilitation prior to selecting the final preservation system. The topics include concrete crack welding processes, hollow plane void repair with a structural welding process, horizontal surface patching, vertical and overhead patching, a pile repairing system for above and below the waterline and the selection of a final protective surface such as deck and pavement epoxy overlayments, build-up epoxy flooring systems and epoxy coating systems.

INTRODUCTION

What is meant by "The Total Concept of Concrete Rehabilitation"? Actually until about five (5) years ago it seemed to be a well kept secret, the ability of making lasting concrete repairs was shared only by a few people who comprehended seven (7) basic rules of knowledge:

1. Understanding the chemical composition, physical properties, limitations and thermal movements of concrete.
2. Understanding why and how deterioration develops in concrete.
3. Understanding the full consequences to the concrete structure connected with a given type of deterioration.
4. Ability to identify the different forms of concrete deterioration.
5. Understanding polymer chemistry, its physical properties, limitations and its relationship when used as a repair material for concrete.
6. Understand proper surface preparation of the concrete to be repaired.
7. Understanding application techniques of polymer chemistry formulas on concrete.

The knowledge of these seven (7) basic rules are required for achievement of durable, lasting concrete repairs. The manufacturer of the epoxy polymer systems must be cognizant of all these rules and the specifier and contractor must understand rule number 2, how deterioration develops in concrete; rule number 3, consequential damage caused by the defect; rule number 4, identification of defects; rule number 6, surface preparation and rule number 7, application techniques.

The Total Concept Approach starts with the identification of cracks. Cracks are most often overlooked or never payed any attention to. Analyzing what happens to the concrete structure, when cracks develop will provide a different viewpoint on how serious the damage typically is. The crack not only breaks through the concrete structure it breaks the cementitious paste surrounding the reinforcement steel running transverse through the new crack void for many meters (feet). Air and water can then transfer through the crack causing accelerated oxidation to the steel and erosion widens the crack by dissolving the water soluble elements from the cement portion of the concrete.

Observation of white or rust colored powder on the surface of a crack is a definite reminder that the crack is being widened because of water movement. As the oxidation process conditions a volumetric increase in the steel takes place creating a larger cavity around the reinforcement steel. In many situations the volumetric change causes additional cracking and spalling. This type of damage constitutes a true structural problem. Property damage caused by leaky cracks to merchandise within the structure from water, dust and insects is only a small part of the monetary loss, the real loss is the structural integrity of the concrete. Typically concrete is not designed to crack. Expansion and contraction joints are designed into structures to take up all the thermal movements.

It should also be noted that when other surface materials such as concrete, asphalt, paints, elastomeric coatings or epoxies are placed over cracks, the crack void will within a short period of time reflect through the newly applied surface material. The reason is that temperature causes the crack to open and close with each thermal change. The only way to stop the crack movement is to weld the entire crack void together creating a monolithic weld as the structure was originally designed.

Flat concrete surfaces such as floors, bridge decks, parking decks and pavements should be sounded for hollow areas within the concrete. Sounding is easily accomplished by pulling chains over the surface. Smaller areas may be sounded using a hammer and tapping the surface. Sound concrete will provide a pinging type of noise, whereas hollow plane areas within the structure will give off a dull unsound noise. It is easy to distinguish the difference in sound and deteriorated concrete. In some instances it may be necessary to core the concrete to determine the full extent and number of layers of deterioration within a slab.

Concrete cracks and hollow plane voids may be welded together utilizing a low viscosity epoxy resin system capable of being applied and cured under normal or adverse conditions of application. The epoxy formula should have superior wetting abilities at lease three (3) times greater than water and be capable of curing under temperature conditions on wet surfaces down to 1°C(33°F) and on dry surfaces down to -18°C(0°F). The welding of cracks and hollow plane areas with a quality epoxy system are normally considered to a permanent restoration solution.

The second phase of "The Total Concept Approach" is to identify all patching requirements. The patching of flat surface concrete is required most often for safety purposes, however, exposure of the reinforcement steel can cause structural stresses leading to future failure of all structures. The importance of strength needs no explanation, therefore, the simple fact that holes in a concrete structure allow moisture transfer, chemical contamination and many times allowing the reinforcement steel to relax could cause severe stress on the overall structure and possible failure by collapsing. Above grade concrete decks requiring mass removal of deteriorated concrete many times require shoring of the structure to maintain the designed tensile and flexural properties of the reinforcement steel until the polymer (epoxy) concrete has developed sufficient strengths to assume its responsibilities.

Overhead patching is extremely important to maintain column and beam integrity. Columns and beams allowed to deteriorate without repair often allow settlement of the upper deck because of the loss of flexural properties.

Patching of flatwork and overhead work should be accomplished with epoxy mortar systems that offer a true structural bond. A system that is capable of creating a mechanical and chemical adhesion to the substrate and a mortar that has similar thermal linear expansion properties.

The third phase of "The Total Concept Approach" is to identify all leaky contraction and expansion joints. Because of time and space requirements this paper will not address the third and final phase.

EXPLANATION OF THE PROBLEMS: CRACKS

Two categories of cracks are of major importance:

1. Surface cracks on flat work.
   - A. Shallow surface cracking often called crazing typically looks like spider webs. They occur when the surface has been overtroweled, broadcasting cement on the surface, incorrect use of vibrating equipment or lack of proper curing. Typically 90% or more of the cracks will only be in the laitance portion of the concrete.
   - B. Random shallow straight cracks often concentrated in the center of flatwork occur when the rate of evaporation is fast, as during hot weather. Typically these cracks are shallow to 100mm (4 inches) deep.
2. Structural cracks in all types of structures
   - A. Random cracks, narrow or wide may occur singly or in groups and can be caused by many factors, including improper joint design or spacing, irregular subgrade settlement, impact damage, earthquakes and the most common cause is shrinkage during the curing process.

VOIDS

Voids within the concrete mass are often early causes of concrete deterioration. Typically the voids are categorized in three (3) groups:

1. Honeycombing is an irregular shaped void, usually in walls or vertical structures, caused by the failure of the cement mortar portion of the concrete to fill the spaces between the coarse aggregate particles. This typically occurs when reinforcement steel is highly concentrated in a given area, vibration has been carelessly done or clearances within the forms have been to small to permit proper vibration.
2. Sand streaking is typically veins or pockets of loosely cemented sand which usually occurs when the form leaks water or cement paste, however, a concrete mix with too much water, too much cement paste or most often a poorly graded aggregate mix causes the problem.
3. Hollow planing is an irregular shaped void running parallel with the surface in flatwork. This damage is typically found just above or below the upper layer of steel reinforcement. The void thickness is determined by the amount of water transferring through the concrete, thermal cycling, freeze-thaw conditions and vibrations on the structure. Conditions which cause the starting of the deterioration within the concrete mass are whipping of air into the mix, poor vibration or too much water in the concrete mix. Hollow planing typically does not become evident for two or more years after placement of the concrete. However, there is a second type of hollow planing which occurs between an aged concrete slab and new placed cover slab. Typically this void occurs from shrinkage in the cover slab

during the curing process or when a bond does not develop between the two slabs.

CONCRETE HOLES

Three categories of holes in concrete should be addressed. Each type of hole could have the same cause or causes for developing the deterioration. Holes developed from porous concrete often occur from bad vibration, poor mix design, honeycombing, hollow planing and liquid water transfer into or through the structure. Actions that speed-up deterioration are thermal change, freeze-thaw and the lack of proper sealing of the concrete surfaces.

1. Shallow holes are most often called spalling. The surface slowly breaks away in small chunks or flakes on flatwork due to excessive traffic. In floors, spalling is frequently noticed first at joints. In pavement and deck areas spalling is typically noticed in areas where the surface was over finished or improper curing of the concrete occurred. Vertical spalls are often caused by concrete forms sticking during the stripping process and water erosion in wet areas. Thermal-movements with heavy concentrations of dew, evaporation and rain or excessive amounts of de-icing chemicals applied to the surfaces in colder climates accelerates the deterioration.
2. Flatwork holes are referred to as potholes. The deteriorating surface may start to erode because of previously mentioned conditions, however, excessive traffic accelerates the breaking of the hole edges thus enlarging the pothole and creating a hazard for travelers. Many bridge deck holes have been identified as starting from hollow plane deterioration. Structural cracks are also considered a major source of pothole development. Typically when the hole becomes deeper than 12.7mm (.50 inch) it is referred to as a pothole, not a spall.
3. Vertical and overhead holes are referred to in general as overhead holes. The deterioration can normally be traced back to mix design or placement procedures, however, weather conditions accelerate any flaw or defect and often vertical damage is noticeable before potholes develop on the surface area. Cracks and leaky expansion joints contribute to the rapid deterioration of concrete structures.

WATERLINE AND BELOW SURFACE DAMAGE

Deterioration of concrete in an aggressive marine environment may occur from wetting and drying cycles, chloride attack, freeze-thaw cycles and polluted water. The same conditions affecting concrete above the waterline affect it at the waterline and below. Cracks, spalling, freeze-thaw and erosion of the surface are the typical types of deterioration, however, impact damage from ships, barges and floating debris are common.

REPAIR OF CONCRETE DETERIORATION - CRACKS

1. Shallow surface cracks called crazing are typically corrected by removing the laitance of the concrete by an abrasive blasting method. These methods include; sand blasting, high pressure water blasting and steel shot blasting. Typically ninety (90) percent or more of the crazing cracks should be lost during the abrasive blast cleaning process and the balance of the cracks should be structurally welded together with the epoxy injection system.
2. Structural cracks running vertical or horizontal require the same treatment. Typically the only difference in flat surface work compared to overhead or vertical crack repairs are the port spacings. Spacings of port locations depends on the concrete thickness and shape. Four (4) factors must be determined before starting the injection work.
   A. Select the epoxy resin by determining the average width of the crack void. Cracks from .05 mm(.002 inch) to 6.35 mm (.25 inch) will require a low viscosity injection resin, approximately 300 centipoises similar to Thermal-Chem Injection Resin, Product No. 2. Cracks between 6.35 mm (.25 inch) and 12.7 mm (.50 inch) will require a viscous injection resin that will stop flowing when you stop pumping. These resins are referred to as an injection gel or known as Thermal-Chem Injection Resin Gel, Product No. 201. Cracks wider than 12.7 mm (.50 inch) could be filled with a sand filled epoxy mortar similar to Thermal-Chem Mortar Resin, Product No. 3. When estimating a crack width, disregard the spalled area at the surface. Try to estimate the actual width below the spalled area.
   B. Selection of the port spacing should be based on the thickness of the slab or wall structure. Often irregular shaped structures require previous experience to determine how the resin will flow. As a typical rule of thumb, the ports may be spaced the same distance apart as the thickness of the concrete up to 304.8 mm (12 inches).
   C. Selection of the type of entry port is determined by two (2) factors:
      1. When the crack has water in it or water is flowing through it an injection tee must be used. The tee is a pre-formed plastic part that covers the crack for about 50.8 mm (2 inches) in length and 19 mm (.75 inch) in width with a raised well area and a plastic hollow tube located in the center of the tee. The tee is set over the crack in a longitudinal direction. Large spalled crack surfaces are more economically set by using tees and placing the tees into the deepest depth of the void and sealing the surface with a sand filled epoxy mortar.
      2. When the crack is dry either the injection tee or injection port may be used. However, most contractors select the injection port method because of the speed and accuracy of intersecting the crack. The port is a pre-formed plastic tube with a wedged shaped flange on the lower portion of the port. The flange squeezes inward as it is being pushed into a pre-drilled hole holding it tight in place.

D. The selection of the surface seal product will be determined by two factors;

1. Wet surface areas are normally difficult for epoxy products because of the fact that they will not setup fast enough to stop the flow of water, therefore, hydraulic cementitious materials are used to hold the tees in place.
2. Moist or dry surface areas are sealed with epoxy bonders. These thixotropic viscous paste type materials are used to hold the tee or port into place and to seal the surface.
3. Surface preparation of the crack depends on the contaminants that are present on the surface, typically most cracks above the waterline require little to no surface preparation. However, there are circumstances when preparations are required; i.e. cracks that have rust or efflorescence on the surface are typically cleaned by sanding or grinding the surface free from the contaminants. Surfaces that have grease or other fatty acid type solutions on them are normally scraped, then abrasive blasted in order to get a clean surface to apply the surface seal material. Seldom are cracks ever cleaned inside the void area. The exceptions are when fatty acids either from vegetable or animal contaminants have permeated into the crack void and could cause poor adhesion of the epoxy to the substrate. In these situations the surfaces are prepared and the ports or tees are set, then just prior to the pumping of the injection resin, a hot water solution of a fatty acid emulsifier, similar to Thermal-Chem Cleaner, Product No. 101 is used at a 2% to 4% solution by weight. The emulsifying solution is pumped through the crack system, followed by a clean, hot water rinsing. It should be noted that this is not a recommended practice, unless the crack is really contaminated with fatty acids. The fear that always develops, especially in fine void cracks is that the loose cementitious materials, broken aggregate particles and dust within the structure could cause a damming effect. When pumping the injection resin into the void, a false impression would be assumed and the crack would not be welded into the monolithic structural bond required to permanently seal the crack. Never under any circumstances use solvents as a flush or cleaning solution.

   Concrete that has not been painted or previously treated with other materials need only be wire brushed and vacuumed. All paint and other materials must be removed down to a clean solid substrate and approximately 25.4 mm (1 inch) on each side of the crack. After removing all foreign materials from the surface, vacuum all debris and dust from the crack. If the concrete has been previously patched, with a band-aid or V-type repair system, tap the surface of the patch with a hammer and if a ringing or hollow sound develops, remove the material. If the material is sound leave it in place and continue on with your crack repair system.
4. Set the entry port or tee at the predetermined spacings. On vertical cracks set the entry port as close to the floor as possible, centered lengthwise over the crack. Set all other ports equally spaced following the direction of the crack. To hold the tee on a vertical or overhead surface, insert a nail through the hole in the tube, on the tee, and hammer the nail into the crack until the nail holds firm. Nails are not required for flat surface applications of tees. For ports, drill into the concrete with a vacuum bit and vacuum chuck attached to a low speed air or electric drill. A vacuum hose attached to a wet/dry vacuum cleaner is then attached to the vacuum chuck. Do not use a masonry drill or star drill to set your ports. The fine dust and particles generated from the drilling process will plug up the crack and give you a false impression of filling the void. Proper vacuum tools are necessary when using the port method. The benefits of using the port method are in the speed and total economy of completing the job. Typically four ports can be set in lieu of setting one tee. The labor savings is fantastic.

   When two cracks come together, no matter what your spacings between the tee or ports are, place an entry port directly over the intersection of the two cracks, even if it means placing the two ports very close together. If you lose the visual sight of your crack after cleaning the surface, do not assume that the crack has disappeared, for when you start pumping the injection resin, the resin will seek out the crack and create a mess by coming out the easiest path of travel at the surface. There are two (2) techniques typically used to locate cracks when the visual sight has been lost: paint the surface with water or blow the surface with dry air. Normally, one or the other method will show the crack. Surface sealing of the crack and entry port is crucial. If a sloppy job is accomplished during this phase of the work, your injection process of actually welding the crack together will become a troublesome and hard job to accomplish. The resin will seek out the smallest pinhole or void and will permeate through those areas immediately. It will also take the easiest path of travel, therefore, the surface of the crack must be thoroughly sealed to prevent injection resin leakage. Apply sufficient bonder around the tees or ports so that only the small tube is seen through the epoxy. Smear the epoxy into the surface of the crack and over the concrete smoothly, creating a tight seal. Allow the surface sealer to become tack-free before starting the welding process.

5. The crack welding process may be accomplished by using three (3) different methods of application, all utilizing the same exact injection resin. The first method is packaged in cartridges whereby the "A" component and "B" component are pre-measured and all that is required is removal of the seal, mixing of the components within the cartridge and placing the cartridge in a hand or air powered caulking gun. The second approach is the oldest method of pumping injection resins into concrete; it utilizes a pressure pot. A small air type, painting pressure pot is used whereby the "A" and "B" components are pre-mixed and placed within the pressure pot. The air pressure is regulated by a regulator and the material pumped through plastic tubes to the entry ports. The third and most sophisticated approach is the injection machine. The injection machine has two (2) positive displacement pumps which properly measure and disperses the epoxy, it controls the temperature of the epoxy resin, viscosity and pumps the epoxy separately to the end of the hoses to a stainless steel mixing tube which blends the two (2) comonents just prior to entering the concrete. Without any question the development of these machines have reduced the total installation cost of structural crack repair work. Both the cartridge and the pressure pot technique require you to be very cognizant of the short pot life of the injection epoxy resin system. The selection of the equipment will be dependent upon the size of the job to be treated. Typically, if more than two (2) liters of epoxy resin are to be used in small void cracks, an injection machine will be selected due to the efficiency and savings of lost materials. If the cartridge or pressure pot method are selected, follow the manufacturer's mixing instructions for the injection system. After mixing the product you are ready to start your welding process. Always select the lowest elevated entry port. If the crack to be repaired is on a flat surface start at the farthest end of the crack and work to the other end of the crack. Once the pumping process is started, you cannot stop your work. Attach the injection resin nozzle to the entry port and start your pumping process at 14 psi for moist or dry cracks and 21 psi plus the head pressure of the flowing water for wet cracks. This is the phase of the work, where patience is required. As you start to pump the injection resin you will feel a back pressure developing on the plastic tube. No matter how slow the resin is moving, you will know when movement is taking place. Pump until the resin oozes out of the next adjacent port. As the resin starts to come out on a moist or dry surface, air and resin bubbles will first appear. Continue to pump until a solid stream of clean resin appears. When water is in the crack, first the water will be displaced, then water and resin and then clear resin will ooze from the port. Once clear resin has oozed from the port, stop the pumping process, plug the port that was being pumped and put the nozzle into the port that just oozed the resin. Never skip a port. Continue the same process until the entire crack has been filled.

On injection projects where the back surface of the structure can be easily sealed, many ports may be pumped at one time. Hold the nozzle onto the first port and pump until resin passes out of the next port. Plug each port as resin oozes out, without stopping the pumping process. Continue to pump as many ports as possible or until a back pressure develops on your plastic tubing that creates a danger of resin of squirting from the nozzle. At that time stop your pumping process, plug the port that was being pumped and move to the last port that oozed resin. Do not skip and go to a dry port. You must pump the one where the resin oozed from the void.

In other situations, it is advantageous to create a manifold, whereby many ports are pumped at one time. Typically, one person can handle three (3) or four (4) pumping tubes on a crack repair project. On a manifold setup, each tube is individually controlled by a turn-off valve, therefore you can move from port to port without shutting off the entire pumping process. Be cognize of the fact that you cannot skip any ports and that you must back track and pump resin between all the entry ports, never allowing a water or air pocket to develop. After a few hours of pumping experience, an applicator can become very proficient at this pumping technique.

Many concrete structures and uneven concrete surfaces, such as waffle-pan construction, where the waffles ribs may vary from 6.35 mm (2.5 inches) in width on the lower side, to 152.4 mm (6 inches) on the upper side with the ceiling/floor being only 63.5 mm (2.5 inches) thick, create real opportunities for the injection contractor. In these situations most of the work will be accomplished from the underside deck area, working always upward. In this manner, you can control the crack injection process, whereas if you pumped from the surface downward, it would be a more flooding type approach which never has worked properly. Pouring a low viscosity injection resin into a crack, will not solve the problem, unless the crack is sufficient in width, i.e. 12.7 mm (.5 inch). For instance, you would not be able to get sufficient material throughout the crack void to weld the crack together. The low pressure is vital to the technique of crack injection. The application technique is as important as the chemical formulation, without both factors failure is eminent. The injection resin must wet the inner surfaces of the void area and create a chemical and mechanical adhesion to the cement particles and a mechanical adhesion to the sand and aggregate particles. The wetting action also creates a channel for the resin to flow through. If high pressures are used, the pumping process often passes by the small fissures and does not travel into the void around the reinforcement steel. It is a known fact that cracks larger than .012 mm (.005 inch) will continue to grow when thermal changes take place. This is why all manufacturers of crack repair materials realize that their materials must permeate to a minimum of .005 mm (.002 inch) in order to prevent future migration of the welded crack.

Hollow plane void repairs are very economical with the structural injection welding process. After sounding the deck and determining the perimeters of the void area, which should be clearly marked with chalk or paint, a grid should be marked across the hollow planed area with spacings of either 304.8 mm (12 inches) or 457.2 mm (18 inches) on center. The drilling process may be accomplished in two (2) steps: First by using a masonry bit to drill within 12.7 mm (.5 inch) of the void area, then a vacuum bit attached to a vacuum chuck should be used to complete the drilling into the hollow planed area. If the masonry bit is used to drill the entire hole the cementitious dust will plug the void area and create a false impression of sealing the void when the injection process is used. Depending on the structures grade and elevation, the contractor has the option of using the port method or just plugging the hole with corks. However, in most cases, in order to control the plugging process and create back pressures, it is most practical and less expensive to use the simple plastic port and plug method. Seal the ports with an epoxy bonder, such as Thermal-Chem Bonder, Product No. 4, and allow it to become tack-free. This requires from thirty (30) minutes to one-hundred eighty (180) minutes depending upon the temperatures of the substrate. Start your pumping process from your lowest elevated port and continue to pump until all water and clear epoxy has oozed from each port. The same port may be used to pump numerous other ports. When the back pressure starts to increase on the pumping tube, stop pumping that port, plug it and move to the lowest elevated port or to a port that would be located in a irregular shaped pocket at the edge of delaminated area. You must be very careful not to allow the air or water to be forced into a void area without an exit port, otherwise you will not fully bond the structure together. On shallow, hollow planed delaminations, that are within 50.8 mm (2 inches) from the surface, a sounding of the deck will determine if you are filling the void completely. As the epoxy fills the void the pinging and hollow sound disappears and the sound becomes solid like the adjacent undamaged concrete. Continue to pump until the entire void is filled. On projects where a lack of bond has developed between two (2) concrete placements, possibly aged to fresh concrete and a depth that exceeds 50.8 mm (2 inches), a sounding will not always be obtained. In these cases coring of the slab may be necessary to determine the size and locations of the voids. It should be noted that all cracks must be welded together during this process and cracks that run through the entire structure must be sealed from the underneath. Traffic may be allowed to drive over the structure during the welding process, as long as all cracks and open voids to any surface are thoroughly sealed with an epoxy bonder. Identification and repairs of hollow planed areas will prevent future damage to the deck and will reduce the maintenance cost over a period of many years. Cores that have been removed from hollow planed injected decks vividly illustrate the movement of resin throughout the structure and magnification of the core verifies that the resin will travel where the naked eye cannot see. The injection process is a simple, however unique, repair tool for concrete structures.

Underwater applications are treated nearly the same as crack repairs above the waterline. The difference is the extra time consumed by the diver in the water and the surface seal materials. Instead of using epoxies underwater, hydraulic quick setting cements are typically used to speed up the surface sealing operation. The pumping process continues in the same manner as a dry surface application. A properly formulated epoxy system will have no constraints in being applied or curing and developing physical properties compatible with the concrete structure underwater. Further on in this paper I will discuss another technique of repairing underwater cracks on concrete structures below the waterline.

REPAIR OF CONCRETE DETERIORATION - HOLES

The successful utilization of epoxy patching materials with Portland Cement concrete results from their physical, chemical and application capabilities with concrete. The systems should provide the following benefits:

1. Safe and easy to handle.
2. Adjustable formulation maintaining 100% solids.
3. Excellent color selection.
4. Choice of normal or rapid cures.
5. Compatible physical properties.
6. Chemical and mechanical adhesion to Portland Cement concrete.
7. Fast strength development.
8. Capable of application, adhesion, cure and development of strengths that equal or exceed that of Portland Cement concrete under normal or adverse conditions of application on moist surfaces down to 1°C(33°F) and dry surfaces down to -18°C(0°F).
9. No special equipment for application.
10. Controlled surface textures.
11. Low installation cost.

The key to the developments of easy handling, improved performance and unique curing abilities are the resin modifications and the curing agent portion of the formulas. Most formulas on today's market cannot meet the above criteria under thermal changing conditions. The resistance of epoxies to chemicals such as: salt water, acid and alkali solutions is well known. The 100% cross-linking structure of the epoxy formula effectively blocks ingress of salt water and other chemicals. Formulations containing volatiles, plasticizers or other materials that could migrate from the formulation or not provide a total cross-link could cause brittleness, poor wearing characteristics or will result in bond line failure. The mortar must contain special properties of surface wetting abilities. Wetting abilities to pre-wet each particle of silica sand and/or aggregate combinations. The same wetting ability must then be used to adhere itself to the concrete substrate to become a monolithic part of the structure. Without good wetting abilities, a patch is doomed from the start.

Particle shape as well as gradation are important factors in developing a balanced epoxy mortar formulation which will properly wear. Sub-angular silica or quartz are recommended for epoxy and mortar formulations. The sizing of the aggregate is crucial, under no circumstances should aggregate exceed one-third of the height of the hole to be filled. Good design dictates a minimum of three (3) of the largest particles of aggregate to stand on top of each other. Epoxy mortar formulations are blended according to the depth of application. The deeper the application the more economical the epoxy mortar becomes per unit, because less is required per unit.Low void content in the mixture is a must, preferably between between 3% and 6%. The aggregate must be sound, clean and dry. The epoxy patching compound should be available in a minimum of two cures; a normal cure should have the application and curing range from 10°C(50°F) to 60°C(140°F) and a rapid cure from -18°C(0°F) to 60°C(140°F). Both cures should provide the same physical properties for mixing the sand and aggregate and provide the same physical strength properties compatible with concrete. The cured physical properties should be such that they never stress the substrate, however, they must outwear the adjacent concrete surfaces.

The success of any patching or overlayment of concrete depends upon the preparation of the concrete surface. Before any epoxies are applied to a surface, the concrete must be sound and clean. Removal of laitance is necessary. The most economical methods of removing surface weaknesses from concrete is sand blasting or high pressure water blasting. Solvents, cleaning solutions and acid etching processes will not take the place of abrasive blasting. Prior to blasting, any loose concrete should be removed with appropriate tools. When grease or other contaminants containing fatty acids have been spilled on the surface, a fatty acid emulsifier should be applied to the contaminated surface and thoroughly rinsed after the abrasive blasting is completed.

All cracks in the pavement or deck should be welded together with a crack repair system prior to patching or overlayment. As temperatures change, the cracks open and close, they will eventually reflect through any overlayment material.

The mixing procedures for epoxy patching materials is very similar to that of cementitious materials. For small quantities, a low speed drill, 200 to 600 rpm, with the appropriate mixing paddle will thoroughly mix the epoxy and sands into a grout or mortar. For larger quantities, a concrete mixer with welded blades inside the drum and a drum that turns as part of the entire mixing action will provide an excellent homogeneous blended mortar. Proper equipment is essential to prevent air entrapment in the epoxy mortar.

It is extremely important to use clean undamaged tools. Hand tools and mixing equipment are typically the same type tools used to place concrete. The mixer should be a mortar type drum unit with three (3) blades welded on the inside of the drum. The drum and blades should rotate as one (1) unit. Do not use a plaster type mixer, whereby the drum is stationary and the blades move independently within the drum. These mixers segregate the larger aggregates and produce an unfavorable mix. Wheelbarrows and large pails are practical containers for hauling epoxy mortars. It is ideal to setup the mixing area as close to the placement area as possible.

After the surfaces have been thoroughly cleaned, a mortar such as Thermal-Chem Mortar Resin, Product No. 3 should be blended until one (1) color develops, approximately one (1) minute of mixing. The mixing ratio of the epoxy formulation should be accurately followed per the manufacturer's directions. Paint or roll the neat material as a primer evenly over the substrate. Do not allow the primer to puddle. An application of approximately .15-.25 mm (5-10 mils) is sufficient. Puddling of the epoxy resin could cause failure during quick thermal change, because of the incompatibility of the neat material in thick mass with the concrete. While the primer is still tacky, mix the appropriate quantities of dry, clean, silica sand and/or larger aggregate into the pre-mixed "A" and "B" components of the epoxy resin and blend for approximately two (2) minutes or until a homogeneous mass develops. The mortar is now ready for placement. Transport it to the application area and apply it immediately. Epoxy mortars should be treated as concrete or asphalt. They require proper compaction and a striking off of the surface in a screeding action to prevent high or low spots. Most applications for patching can be finished with a screed. Hand trowelling is only required when an existing concrete surface adjacent to the patch is smooth. When the mortar becomes tack-free, approximately 60% to 80% of its physical strengths will have been achieved and at that time pedestrian or vehicular traffic may use the area. Tack-free stages are dependent upon temperature. Substrate temperatures around 0°C.(32°F) require approximately three (3) hours time and at 10°C(50°F), approximately one and one half ($1\frac{1}{2}$) hours, at 20°C(68°F) approximately one (1) hour and at 32°C(90°F) approximately one half ($\frac{1}{2}$) hour.

Vertical and overhead patches may be installed in two (2) manners, shallow areas are typically hand trowelled and deeper voids are formed when practical and the epoxy mortars are pumped or poured into the void.

After the surface preparation has been completed, prime the surface with a low viscosity epoxy resin similar to Thermal-Chem Mortar Resin, Product No. 3. However, the mortar resin used in the mix must be a high viscosity type material referred to as a gel, a product similar to Thermal-Chem Mortar Resin Gel, Product No. 304. A select blend of silica sands will be added to the pre-mixed "A" and "B" components of the epoxy mortar gel and blended until a homogeneous paste consistency is developed. A good epoxy mortar should be able to be trowelled overhead or vertically in approximately 25.4 mm (1 inch) thickness. Typically if the void is deeper than 50.8 mm (2 inches) an estimate should be made to determine which is the most economical approach, forming or hand trowelling? All deeper voids normally are formed and poured. It should be noted that the typical finisher, capable of finishing epoxies or concrete on flatwork cannot finish overhead or vertical applications. A person with a plastering background of cementitious material is the ideal candidate for this type of work. Do not apply solvents to the surface of the epoxy to smooth the surface area. If the epoxy mortar is not finishing correctly, most likely the sand gradation is wrong. Correction in the gradation will eliminate rough surfaces and sagging conditions. Temperature plays an important part of overhead and vertical trowelling. The aggregate must be adjusted as the temperature rises, because the higher the temperature the lower the viscosity of an epoxy and this adjustment must be made in the field. Trowelling of the mortar is extremely important, for all the compaction must be accomplished by pushing of the trowel. In situations, of limited space, the application will be applied in a 12.7 mm (.5 inch) layer leaving the surface rough and then immediately applying the second 12.7 mm (.5 inch) layer. Do not try to hang more than 25.4 mm (1 inch) of mortar per total layer because sagging will develop. When a finished layer is to be applied over a previously applied tack-free layer, leaving the surface rough allows for easier hanging of the next layer. It is necessary to prime each layer of material after it is tack-free so that the next layer can have proper adhesion. All layering can be accomplished as soon as the tack-free stage develops, however, do not wait longer than 24 hours after application of the last layer of epoxy mortar.

Forming is often the most practical method of repairing columns and beams. All surfaces must be properly prepared to sound concrete before setting the form in place. The typical form material used is wood covered with polyethylene. Form oils and release agents do not effectively work with epoxy formulations. Pre-form your area and seal the edges with a thin piece of foam to prevent leakage of the epoxy resin from the surface. Tight fitting forms are extremely important, because if they move during compaction, a failure will occur in the patch. When epoxy mortars are pumped, no priming of the surface is required because there is ample free resin in the mix. However, on extremely rough surfaces pre-priming of the area is recommended. Pump or pour the mortar into layers approximately 152.4 mm.(6 inches) deep. Compact, pour the next layer, compact, and continue the process until the void is filled. A quality designed epoxy mortar formulation for pouring or grouting should never have an excess of free resin on the surface. The sand aggregate should be evenly mixed on the surface as it is throughout the entire blend. After the patch has become tack-free the forms may be removed. When forms are used, a slick thin surface of epoxy resin is very apparent on the exposed areas. To remove the shininess, coat over the area or use a grinder and remove the thin film. If decorative purposes are not required it could be left just as pulled from the form.

Patches for spalling, potholes or overhead deteriorated areas, when properly applied, may be considered a permanent part of the structure. A quality epoxy mortar will outlast high quality concrete three (3) to four (4) times.

REPAIR OF CONCRETE DETERIORATION - MARINE ENVIRONMENTS

The successful repair of concrete, wooden or steel piles above or below the waterline will typically include one (1) or more of the following:

1. Structural crack repair.
2. Permanent patching.
3. A protective jacket system filled with epoxy to prevent furture deterioration.

The successful utilization of epoxy materials in water environments is unique. Over the years many epoxy formulations have been introduced to the market, however, because of their poor curing characteristics or adhesion properties, the systems have never really solved the problem of structural repairs. During the past five (5) year period structural epoxy technology which is both an art and a science has been coupled together with years of application experience in developing modified epoxy formulations. Many of these formulations will cure under cold, submerged and adverse conditions of application. Combining these types of formulations with an injection jacket, it is now possible to not only encapsulate a pile, but to provide new physical strengths that restore the integrity of the pile, without creating a dead load on the structure. The jacket system is utilized in many different types of repair applications. In areas where erosion has taken place and only a thin area of epoxy is required for restoration, a jacket that totally isolates the deteriorated area can be injected with a low viscosity resin similar to Thermal-Chem Injection Resin, Product No. 2. Areas requiring larger quantities of epoxy are filled with a underwater sand filled epoxy mortar such as Thermal-Chem Underwater Mortar Resin, Product No. 312. Either one of these systems will totally encapsulate the concrete pile preventing any further damage and adding new integrity to the structure. However, there are situations where even more structural integrity is required, i.e. a pile that developed a compressive crack during placement. The damage could be meters (feet) below the surface requiring an injection process that will totally weld the crack together, displacing all the water within the crack. Such a jacket has been developed that creates sections within the jacket referred to as positive and negative sides and is pumped full with a low viscosity, high wetting type resin such as Thermal-Chem Injection Resin, Product No. 2. The encapsulation repair method offers a superior way of repairing and preserving wood, steel and concrete piles. The jackets can also be shaped to any form. The versatility of the epoxy formulas allows application and curability of the material down to freezing temperatures. The fast easy installation of the jacket is partially accredited to the assembly of the jacket above the waterline. The jacket is then fitted around the structure and then dropped into position requiring the diver to adjust the location and tighten the jacket by pushing the removable handle inward. The handle is removed after the permanent locking bolts have been tightened. Quick couplers provide easy attachment of the injection manifold for the pumping of the injection resin from the surface. Pumping of the resin is completed in a short time and by utilizing low injection pressures all the water from inside the jacket is displaced. The color of the jackets are white to aid the diver in brackish or dark water installations, however other colors are available upon request.

The factory installed compression seal on the top and bottom of the injection jacket prevents the loss of epoxy during the pumping process. The seal is compressed during the adjustment stage. On jackets that are to be pumped with epoxy mortars the top seal is left off thus reducing the need for entry and exit ports. As the epoxy mortar displaces the water from from within the jacket area, the mortar becomes the permanent seal on the top of the jacket.

The structure is protected by two (2) superior elements. On the exterior the injection jacket is a specially formulated ABS plastic providing a resistance to salt, acids and other pollutants including erosion from sand and waves. The interior of the jacket is filled with epoxy injection resin which adheres to the inner surface of the jacket and the substrate of the pile, thus forming a monolithic structural bond. The injection jacket system provides compatible compressive and flexural strengths that equal or exceed that of the original design substrate material.

The injection jacket permanently becomes part of the structure creating a protective shield. No other supports or attachments are needed to prevent it from falling off of the structure. The injection jackets are lightweight and easily handled by one (1) person reducing extra labor on the project. Standard sized jackets are available in round diameter sizes of 305 to 666 mm (12 to 26 inches) and square jackets are available from 152 to 584 mm (6 to 23 inches). Larger sizes and special shapes are available upon request. There are no limitations to the length of the jackets as they can be combined one upon the other in an easily assembled method. Most jackets are pre-assembled above the waterline by laborers reducing the divers time in the water. The entire system has been designed to utilize a minimum amount of labor reducing the overall cost of the project.

FINAL PROTECTIVE SURFACE SYSTEMS

After all of the structural repairs have been corrected, crack repairs, patching, contraction and expansion joints, the surfaces are now ready for a long term protection program. These programs will provide the necessary waterproofing, abrasive and corrosion protection. In many applications, decorative protection may be achieved, however, under most circumstances decorative applications are secondary because of the natural and chemical exposures. Selection of the correct surface protection should include the following considerations:

1. Resistance to corrosives.
2. Estimated life.
3. Solid content.
4. Thickness of the system.
5. Temperature condition of application.
6. Ease of application.
7. Ease of repairing damaged areas.
8. Total applied cost.
9. Cost per square meter (sq. footage) per year of service.

The above considerations will serve as a guide in making one (1) of the following selections for any given concrete preservation project.

DECK AND PAVEMENT OVERLAYMENTS

Epoxy mortar systems are similar to that of epoxy patching systems. The two (2) component epoxy resin system is blended with a select gradation of silica sand. When properly formulated they will provide superior protection for vehicular traffic, chemical and industrial floors and decorative pedestrian walkways. A tough resilient epoxy mortar should exhibit total waterproofing and natural environmental exposure abilities. Many epoxy mortars are capable of withstand ing high acid solutions in heavy industrial and chemical environments. A typical epoxy mortar should extend the wearing surface of quality concrete by at least four (4) times.

BUILD-UP EPOXY COATING SYSTEMS
Build-up epoxy coating systems consist of a two (2) component epoxy formulation with a graded silica sand seeded into the epoxy system. Concrete structures that are exposed to light chemical exposure, natural environments, low speed vehicular traffic on parking structures and tow motor or forklift traffic conditions in factories enjoy the saving benefits of not applying an epoxy mortar system. These systems reduce the cost of an epoxy overlayment considerably and provide maintenance free repairs for years of normal use. The systems are normally only about 35% of the normal thickness of an epoxy mortar overlay, however, abrasion testing has verified that the system will outwear quality concrete by at least two times.

EPOXY COATING SYSTEMS
Two (2) component epoxy coating systems provide excellent protection to concrete surfaces such as water tanks, sewage holding vessels, flooring systems, and waterproofing applications. The epoxies are available in penetrating sealers to preserve brick and exposed aggregate concrete structures and 100% solid, hihg film build systems to withstand corrosive exposures. The coatings provide a glossy, tile-like, dense finish which is easy to clean. Microbacteria and fungi growths will not sustain themselves on the coating. Epoxy coating systems will extend the life of the concrete many times if not used in areas where heavy abrasive traffic conditions exist.

CONCLUSION
Based on years of experience in formulating epoxy polymer systems for the preservation and rehabilitation of concrete structures, the total concept of concrete structural rehabilitation is mandatory if any length of time is to be expected from the final protective surface system selected. Structural epoxy technology, which is both an art and a science, enables todays architect, specification writer, engineer, contractor or builder to select systems with performance characteristics and durability that can be matched to almost any set of simple or challenging circumstances. However, for lasting results it is strongly recommended that a manufacturer's representative, who has been thoroughly trained by the factory, help select the materials which will allow the contractor to perform simple and routine applications. The manufacturer's representative should be requested to assist you by providing the proper specifications for material and application techniques, thereby reducing the total cost of installation.

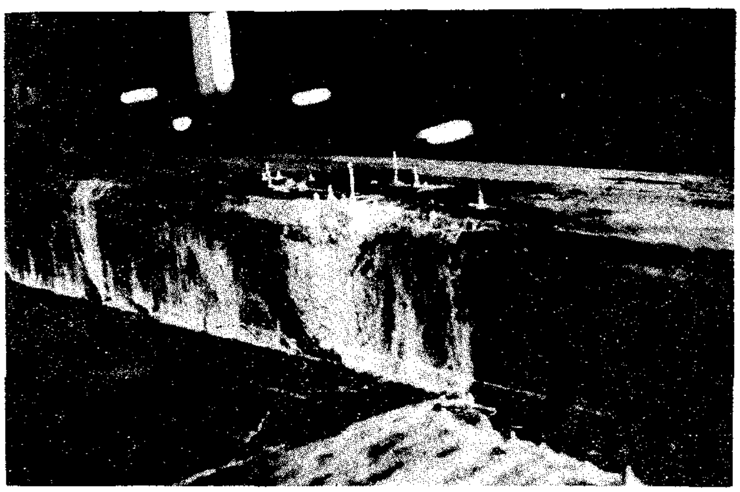


Photo No. 1

Concrete cracks allow the water to move through the crack void which causes the soluble particles of cement to dissolve forming salts on the surface of the concrete structure and the moisture has exhilerated the corrosion of the steel reinforcement causing pressures to be exerted on the concrete and forcing small pieces of concrete loose at the top of the beam.

Photo No. 2

A small hole caused from chirping has continued to grow to a major pothole. Freeze-thaw damage exhilerated the original problem and now traffic will continue to break off the edges of the pothole increasing the size of the hole everyday.

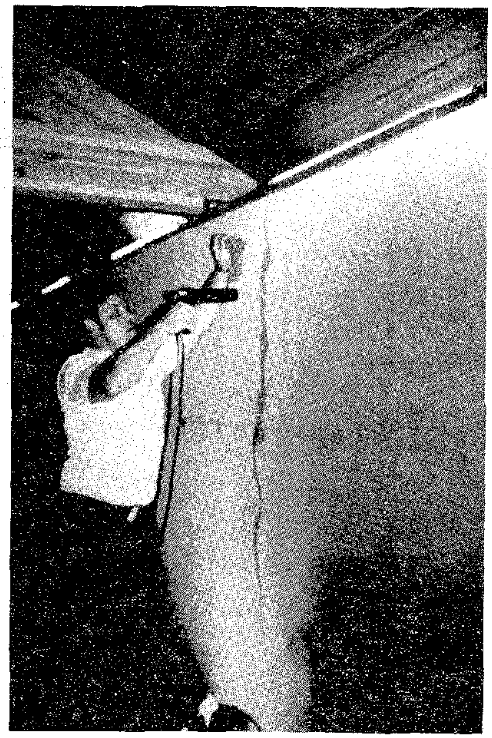


Photo No. 3

Crack repair is sufficiently accomplished by pumping a low viscosity 100% solid epoxy injection resin into the crack. The epoxy welds the entire crack into a monolithic structure preventing any further air, water, or insects from moving through the crack.

Photo No. 4

A delamination just above the reinforcement steel called hollow planing, is being welded together with a two component 100% solid epoxy resin system. The repair will be considered permanent.

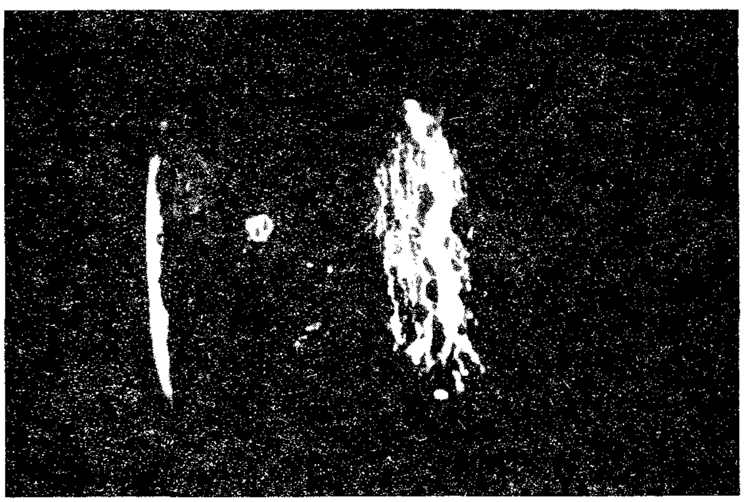


Photo No. 5

A core removed from a concrete slab that was delaminated above the second layer of reinforcement steel. The white lines indicate the passage of epoxy resins; many measurements taken from this core were less than .002" in size.

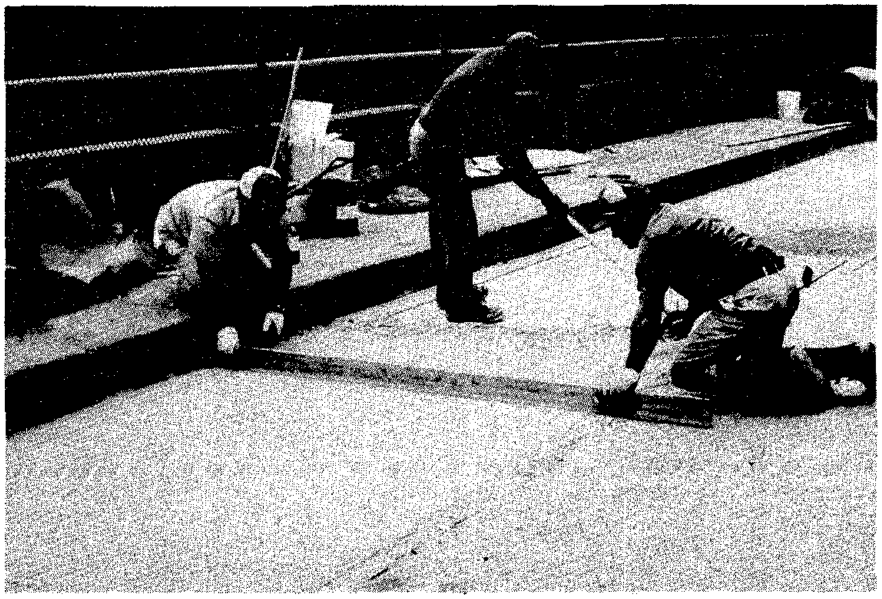


Photo No. 6

Bridge deck patching in overlayments are easily accomplished with two component epoxy mortar systems. This photo illustrates the striking off of a patching overlayment system being accomplished in one (1) operation.

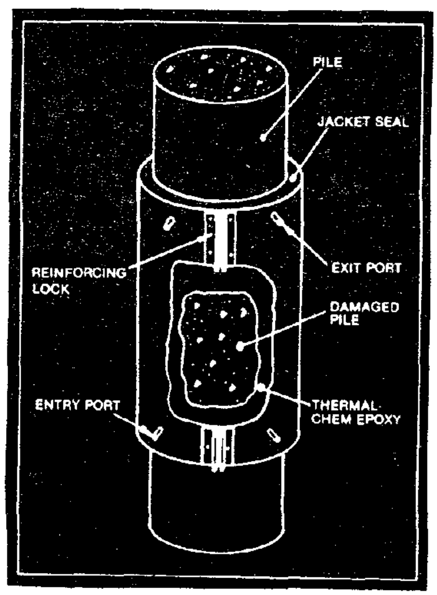


Photo No. 7

Vertical and overhead patching is easily accomplished with epoxy mortar gels. The above photo shows typical damage to a bridge structure when water is allowed to move in and out of the structure freely. The structure is first cleaned, primed with clean epoxy and the mortar gel is pressed into place in minimum one inch layers. Patches such as this should be considered permanent. The above drawing illustrates a typical jacket system with the repairing of wood, steel or concrete piles. The jacket acts as a form to hold the epoxy in place and then is left to provide additional protection from erosion and other damaging elements.

CAUSE & CURE OF

DISTRESSED STRUCTURAL CONCRETE

T.Z. CHASTAIN

Concrete as a structural material differs from all others; it does not come ready-made to the site; it is usually manufactured from nearby materials; it is handled, placed (and sometimes poured) and shaped by local labor; and, concrete is prone to misbehaving if anyone of the many controls is either improperly implemented or disregarded. A witty description of the necessary precautions appeared in the Engineers Journal of Ireland in 1966 under an appropriate heading.

"THE LADY IS FOR MIXING"

"She requires coddling and wooing, she requires all manner of treatment and courtship before disclosing her qualities; she gives off warmth; she shrinks; she is fickle; she is the subject of books and books. She is a good mixer and she requires constant supervision.

"Above all - given proper consideration she ages gracefully.

"Concrete is a lady.

"The make-up of any lady may involve the artful use of creams, lotions, powders and perfumes. They can impart qualities of mystery, attraction, allure.

"The choice of admixture depends, one feels, on an equally rational decision. Regrettably, however, milady concrete must rely on her progenitor for the choice of that which will make her shrink or expand, retard or accelerate, be strong, resilient, dustproof, water repellent, plastic - even lovable.

"The formulae which confer these infinitely desired properties, are, as with the cosmetics, closely guarded secrets. Progenitors of concrete, be they architects or engineers, may be told in strict confidence part at least of the secret, but only on request.

"So concrete is a lady but ....

"She may be a bitch betimes, but if she has quality she requires little addition or embellishment."

This paper is mostly about the bitch and partly about the preventative medicine necessary to retain milady's quality.

Certain basic principles are common to the repair of all types of reinforced concrete structures. The first signs of deterioration are usually fine cracks and rust stains which may be accompanied by spalling of the concrete. While cracking may be due to other factors, the rust stains and spalling are caused by corrosion (rusting) of the reinforcement. The same signs of deterioration can occur in marine structures except that physical damage can be caused by wave action. With some structures, there may be chemical attack on the concrete itself, and in extreme cases, this can cause rapid deterioration.

A skin rash to milady is rust stains and spalling of the concrete which is caused by corrosion of reinforcement before the attack on the steel has progressed very far. This is because corrosion products occupy a larger volume than the original steel and the expansion which accompanies the formation of the rust causes cracking and spalling of the concrete cover. Thus, the evidence of sickness in milady is detected at an early stage and long before deterioration has progressed far enough to knock milady off her feet-in other words, endanger the stability of the structure.

The usual causes of the deterioration of the health of milady are inadequate cover of the reinforcement poor quality concrete, thermal contraction and/or shrinkage cracks which have been allowed to remain without repair. These defects are, in a few cases, aggravated by the presence of calcium chloride in concrete the chloride ions being a contributory factor in the corrosion of the reinforcement. The existence of chlorides in the concrete can, in certain circumstances, seriously reduce the durability of any repair work.

The basic steps of a concrete repair program may overlap but, in general, they are:

1. Condition Survey
2. Cause of Condition
3. Feasibility
4. Selection of Method
5. Scheduling
6. Preparation
7. Application

Step 1

The condition survey includes a careful visual examination supplemented by pachometer tests (electromagnetic type cover meter) to locate reinforcement and determine the concrete cover, and electrical half-cell potential tests (ASTM C876) to determine corrosion activity on reinforcement. All findings should be recorded on plan and elevation drawings of the structue.

Figure 1 - Chain Drag to Determine Delamination by Sound

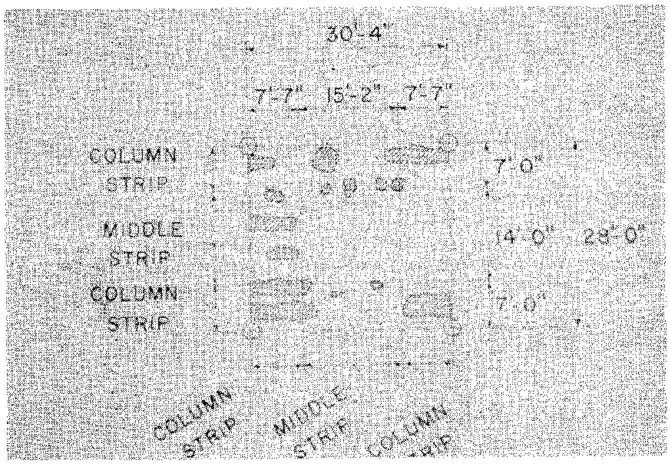


Figure 2 - Record Delamination Location on Plan

Step 2

The cause of the condition requires taking samples for laboratory examination and testing. Depending upon the findings in Step 1, tests of all ingredients may be required, as well as a petrographic analysis and potentiometric (electricometric) tests to determine chloride content. In like manner, the performance of on-site tests, including resonnant frequency and pulse velocity tests to determine the extent of cracking, any be required for some members.

Figure 3 - Locating Reinforcement by Pachometer

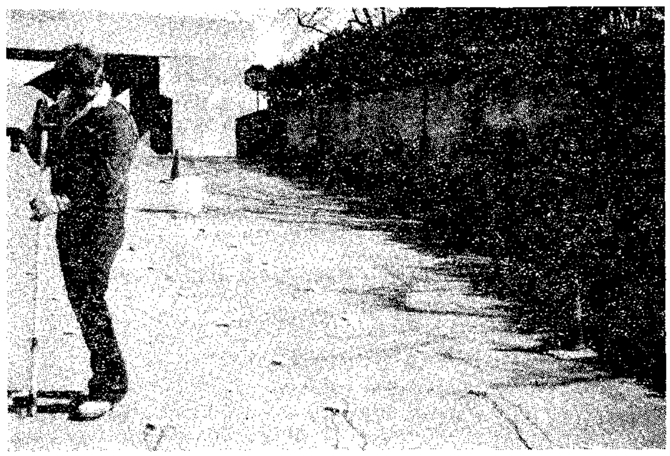


Figure 4 - Electrical Half-cell Potential Test to Determine Corrosion Activity on Reinforcement.

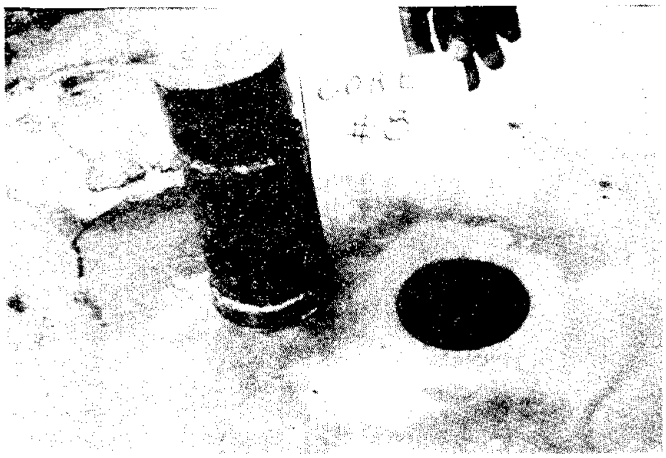


Figure 5 - Delamination Due to Corrosion of Reinforcement

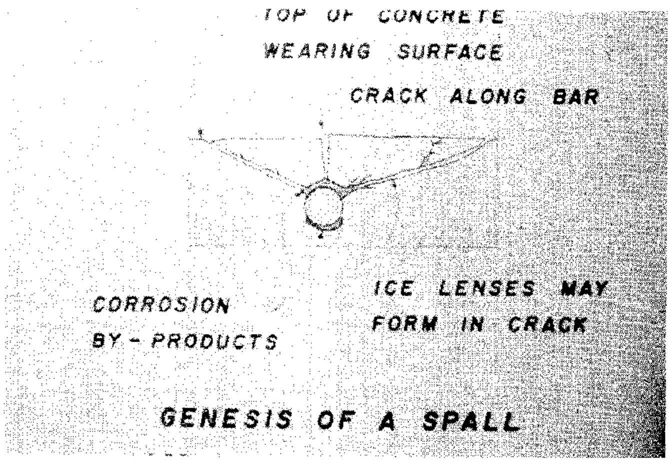


Figure 6 - Genesis of a Spall

Figure 8 - Petrographic Analysis

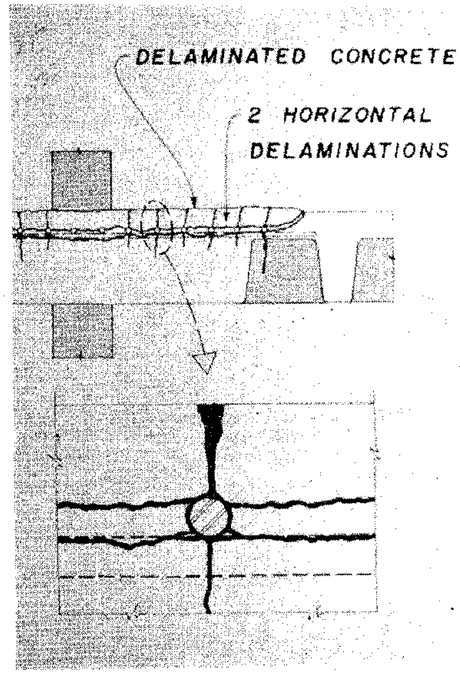


Figure 7 - Genesis of Delamination

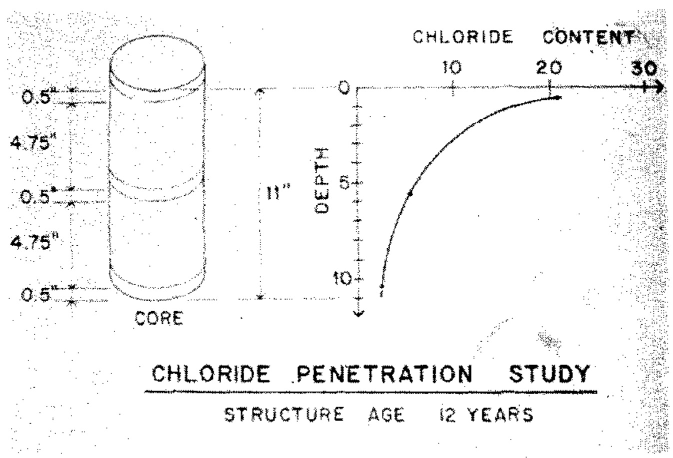


Figure 9 - Chloride Penetration Study

To supplement the information already obtained, it may be necessary to make an examination of the architectural and engineering drawings and depending upon the circumstances, it may be helpful to interview the original general contractor and material suppliers. For practical purposes, failure and defects in concrete structures can be placed in five main catergories;

1. Structural deficiency resulting from such causes as error in design, errors in construction, impact, explosion, seismic activity, and change of use resulting in higher live loading than was originally allowed for in the design.

2. Fire damage; this often results in some weakening of the structure as a whole, as well as severe physical damage to the individual concrete members (floor slabs, beams, columns, etc.)

3. Deterioration due to poor quality concrete, inadequate cover to reinforcement, and the presence of chlorides in the concrete.

4. Chemical attack on the concrete and/or reinforcement.

5. Physical damage caused by the use to which the structure or part of the structure was put, such as the abrasion of a floor slab in a factory, and the abrasion of silos and hoppers holding coarse granular material.

Step 3

In most cases the feasibility of the repair both from the use and economic standpoint to the owner can be determined before reaching this point; however, there are times when it will be necessary to proceed with the next step before a decision can be effectively made. It could be that the damage is such that the structure cannot be repaired at reasonable cost, or with any assurance that the repair will be permanent.

Figure 10 - Parking Deck Beyond Repair

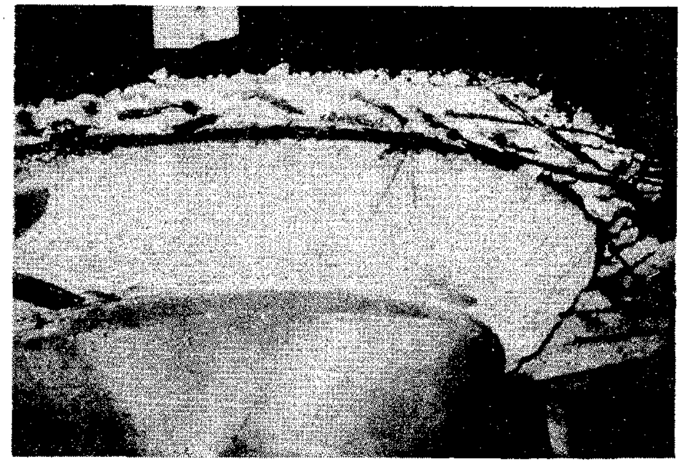


Figure 11 - Column Stripped of Deck

Step 4

Selection of method depends on the nature of the distress, adaptability of the proposed method, environment, availability of materials, costs, desired appearance, and whether the repair is to be a temporary expedient or a permanent restoration. To predetermine the quality of the repair, sometimes a sample repair is made either on the structure or in the laboratory.

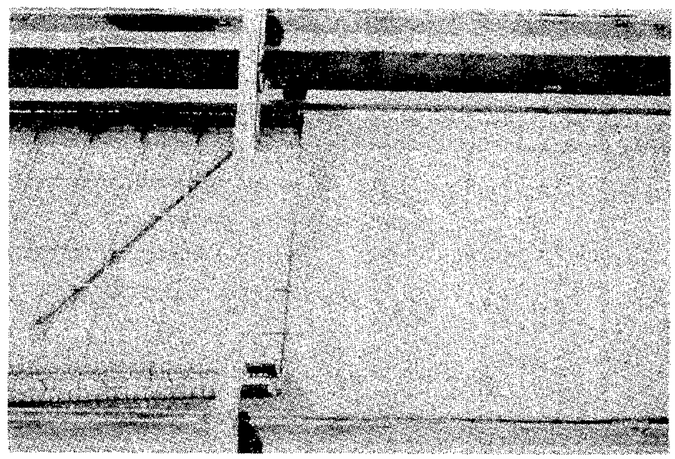
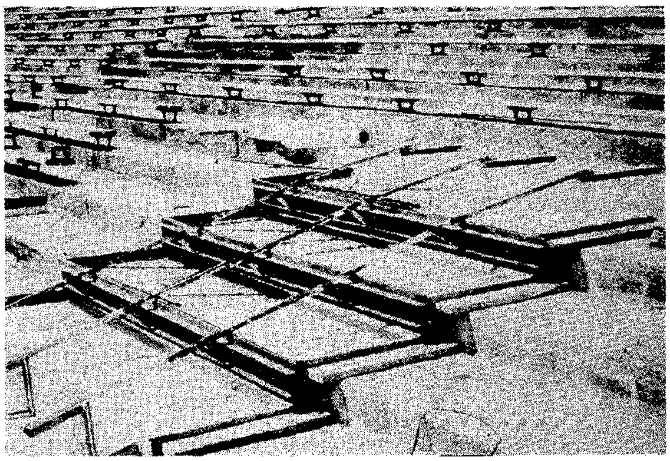
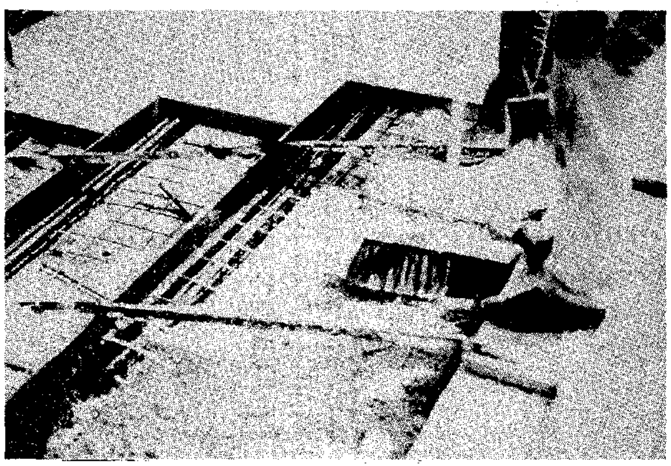


Figure 12 - Feasibility Study of Latex Concrete Repair of Soldier Field, Chicago

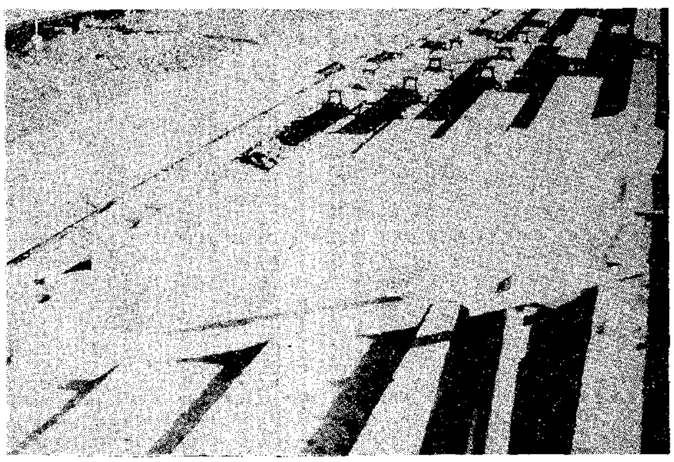
a. Sand Blasted Test Area



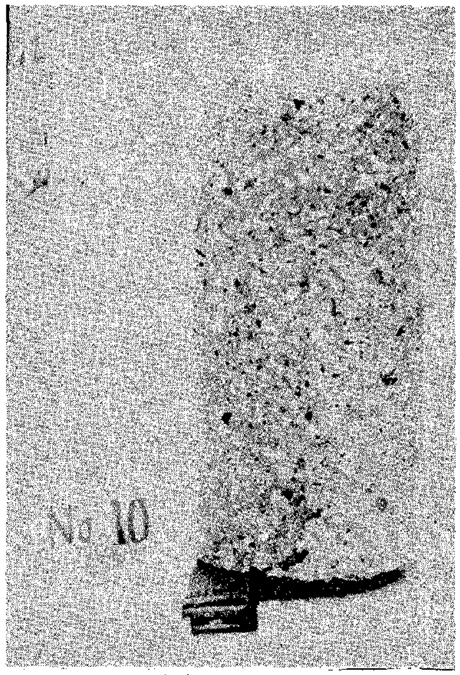
b. Forms with Reinforcing in One-Half of Test Area.



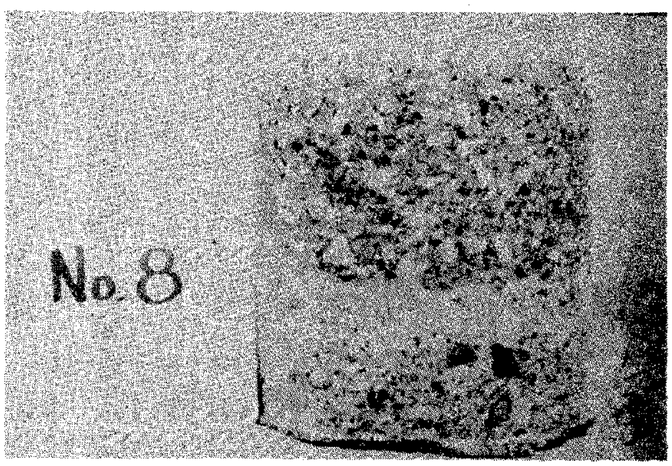
c. Brushing Bond Coat Before Placing Latex Concrete Test.



d. Curing Latex Concrete Test.



e. Core through Riser-Bond Good



f. Core through Tread-Bond Good

Exposure conditions can have a major impact on the kind of materials. For example, special aggregate may be required for a surface subject to abrasion.

With several minor variations, the primary procedures are:

1. Concrete replacement - used for major restoration of a large volume of concrete.

2. Preplaced aggregate - similarly used for major restoration, especially over a large area, such as dam spillway.

3. Shotcrete - for building up an area in layers, especially for restoring large areas of spalled, eroded, or similarly damaged surfaces.

4. Drypack - is used for minor filling of deep, small voids of small cross section. It is especially adapted to filling form bolt holes, some cracks, and similar hollows.

5. Chemicals - include many forms of epoxy resin, other resins, and latexes. Their principal uses are for filling cracks, sealing joints, plugging leaks, surface coating and sealers, adhesives, and certain structural applications.

6. Plaster - is a Portland cement-sand mixture applied in a thin coating by hand or machine for cosmetic purposes. Thickness is usually 1 in. (25.4 mm) or less. Plaster has no structural value.

7. Overlap - can be classified as concrete replacement of a thin layer on the surface, usually of a pavement, floor, or similar structure. Thickness is usually between 1 and 4 in. (25.4 mm - 102mm).

Step 5

Scheduling involves priorities based on weather conditions, job progess, and the need to get the structure into a serviceable condition as soon as possible. Repair of construction-related defects in new or immature concrete should be accomplished as quickly as possible to assure a sound restoration. Repairs to concrete that has suffered from exposure or overloading will probably continue over a protracted period of time. Repairs of this nature can take on the characteristics of a major construction job requiring detailed flow diagrams to coordinate all aspects of material requirements, personnel, equipment, and work assignments to fit seasonal weather conditions and service demands on the structure.

Step 6

Preparation for the repair can now be accomplished . This may be as simple as cleaning loose material and grease out of a form-bolt hole, or as complex as the major preparation of a dam spillway or pilings and beams beneath a waterfront wharf. It is obvious that the kind of distress we are attempting to correct has a major influence on the preparation. Cracks require different preparation than scaled areas.

Where there is unsound or disintegrated concrete (and this includes rock pockets) the first step is removal of all unsound concrete. It is at this point that the groundwork is laid for a durable repair, or a patch that soon loosens and fails. All loose, unsound, soft, disintergrated concrete must be removed down to bright and sound, undamaged concrete. If there is any doubt about an area, more concrete should be taken out. After this has been accomplished, the area is thoroughly cleaned with water, compressed air, or air-water jets, and an inspection made.

Damaged reinforcing steel or inserts should be replaced and, in some cases if required, additional steel inserted. Special fittings will be necessary for some types of repair. Large replacements, pre-packed, or shotcrete repairs may require the construction for forms or molds. All of this now leads to the final inspection and approval of the repair area to permit the application of the selected repair procedure.

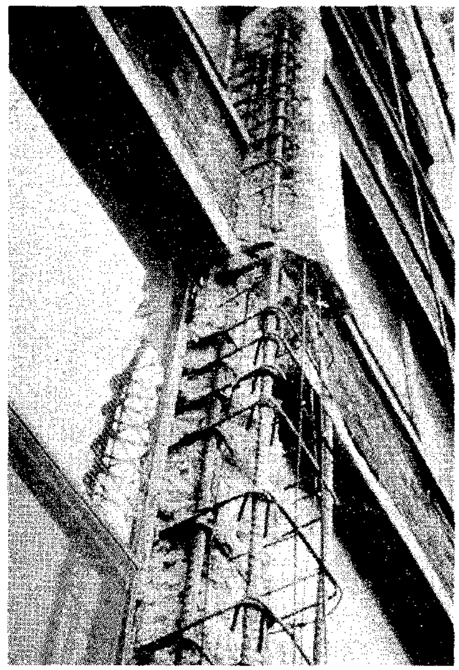
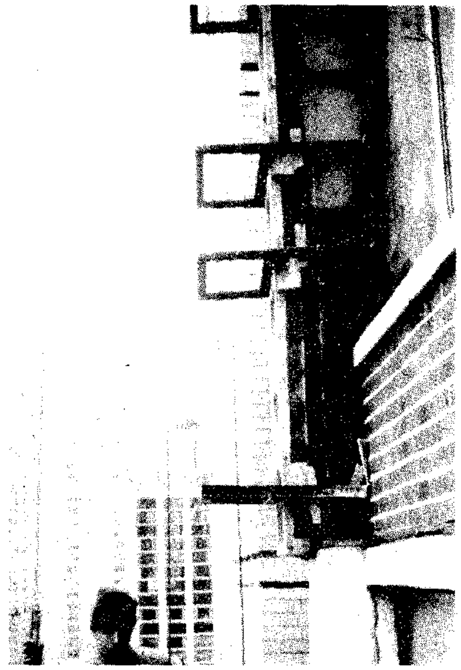


Figure 13 - Column Repair

a. Unsound Concrete Removed

b. Cleaning by Sand Blasting



d. Special Forming

c. Epoxy Coating on Reinforcement to Eliminate Continued Chloride Attack

Step 7

Application procedures involve first the proportioning of the fill material and selection of the tools and equipment necessary for the proper and expeditious completion of the repair. This is followed by actual placing or application of the fill, done in accordance with construction practices. Proprietary materials require close adherance to the manufacturer's instructions. Special attention must be given to matching the color and texture of the existing surface where the surface is exposed to view. In many structures color and texture of the surfaces are not important as long as a workmanlike, uniform appearance is achieved. Finally, to assure the optimum benefit from the repair, care must be used to provide curing and other protective measures. Manufacturers of proprietary materials provide instructions for protection and aging their products.

Credit - Photographs by Wiss, Janney, Elstner & Associates, Inc., Northbrook, Illinois

TOWARD
QUANTIFICATION OF ECONOMIC AND OTHER SYSTEMIC IMPACTS OF BUILDING REHABILITATION AND URBAN RENEWAL

DR. J. K. SRIDHAR RAO AND M. G. NENE
PROFESSOR OF CIVIL ENGINEERING, CALIFORNIA STATE UNIVERSITY
LONG BEACH, CALIFORNIA 90840

CHIEF STRUCTURAL ENGINEER
FLUOR POWER SERVICES
IRVINE, CA. 92730

ABSTRACT:*

The objective of this paper is to attempt to develop a quantitative methodology for impact analysis of building rehabilitation and urban renewal projects, using stochastic (probilistic) resource-based methods. One of the approaches would be to select only those projects which maximize the probability of social benefit for each system (economic cost, adaptability for re-use, desire for housing, equity consideration for minorities and old people, esthetics, energy conservation through transportation mobility, increased economic activity in neighborhoods) and environmental criteria (urban blight, safety, law enforcement, social benefits, political acceptability, etc.). The methodology was developed by Smith for highway corridor selection and by the author for public transportation project site selection. It would be developed for building rehabilitation projects to help to identify and to demonstrate need in face of detiorating environmental damages and to use the information developed in NBS-(CBT)-HUD study of economics of abandonment and rehabilitation (Ref. Colwell, et. al.), the "Costs of Urban Sprawl" study by Real Estate Research Corporation, "Compact Cities" by G. B. Dantzig and T. L. Saaty, et.al. This system methodology helps policy planners, decision makers and project evaluators as well as building regulation agencies.

Building rehabilitation projects need to be looked at not only from the cost-effectiveness viewpoint in an inflationary economy, but also in terms of positive environmental consequences of urban renewal, community development, energy conservation, reduced demand for suburban travel, thereby reducing demand for gasoline and other scarce non-renewable resources. At the same time, for the system to have safety, serviceability, and durability; considerations of margin of safety against seismic, fire, wind and other disasters through strengthening, need to be done. Since there is competition for financial, material (renewable and non-renewable) resources the problem of rehabilitation strategy vis-a-vis new housing developments need to be evaluated with economic (primary and secondary), social, political and economic impact assessments. In many cases where there is a competition among many building rehabilitation and urban renewal projects in the same geographical area, there is a need to have a framework for evaluating alternative rehabilitation projects subjected to objective as well as subjective criteria.

This proposed methodology uses an integrative Resource-Economics-Energy Conservation-Environment-Problem-Solving (R-E-E-E-EP) approach for assessing impacts of building rehabilitation projects for decision making. It uses probability concepts as a unifying measure for combining subjective (judgement) and various objective measures involving multiple, complex, project evaluation criteria.

*This paper is in Abridged form because of time constraints. The methodology proposed in this paper and the views expressed are those of the authors and not the organizations employing them.

INTRODUCTION

There is considerable evidence of the growth of interest in existing buildings in U.S. since 1970s. Most American cities has one or several areas that are undergoing such private renovation.

At present codes are applicable in the case of
1. Change of occupancy or 2. the value of alteration or damage repair work to be accomplished exceeds certain designated limits or when the building is expanded. Present Codes and Standards provide more safety and health for the building user.especially in seismic areas. However, building codes may be extremely conservative in terms of design strengths, loadings for engineered buildings while for unengineered buildings they may not be conservative. However, the criteria needs to be a priori mentioned as for as possible in performance oriented quantitative measures. The impacts of various criteria need to be evaluated to choose appropriate criteria for rehabilitation projects especially when there is a choice between rehabilitation and urban blight, by doing almost nothing. For example, the economically realistic building codes of Los Angeles for seismic strengthening of hazardous unreinforced masonry buildings for life safety may be preferable to the more stringent criteria of limiting damages (architectural, structural, economic).

Historical monuments, damaged structures and those to be remodeled or the use of which is modified, often pose the problem of deciding about adequate safety levels and compliance with current building codes. In some regions, large portions of important buildings have been designed and built according to standards that were afterwards deemed insufficiently strict, and there are large numbers of unengineered dwelling units. Adoption of standards applicable to new structures is cumbersome and expensive in most cases mentioned above. The situation must be coped with having in mind that the objective of engineering design is to limit damages as well as maximize life safety thus optimize for society. Decision models dealing with these cases have recently been developed by Rosenbleuth.

However the existing codes and other regulations are primarily addressed for new construction and rigorous application of these codes have a larger impact on rehabilitation in terms of 1. larger project costs, 2. disruption or destruction to the building fabric, 3. replacement of serviceable materials.

There is a growing adoption and use of Historic preservation waiver clauses in Building Codes and 2. Administrative regulations contained in historic distric legislation. Studies need to be conducted to determine the effect of these codes and regulations. The impacts of building rehabilitation vis-a-vis reconstruction in suburbs or tearing down/leaving it to be dilapilated thereby contributing to urban blight need to be looked at a systemic impact level which includes economics. Thus economy of scale needs to be viewed in conjunction with economy of integration. A methodology for systemic impact analysis of building rehabilitation and urban renewal projects in terms of (1) functional systemic resource (2) positive environmental and (3) remaining adverse environmental impacts is attempted in this paper.

General Economic Considerations for Building Rehabilitation

Building rehabilitation may be performed if any of the following economic principles (Ref. 2) are applicable for the particular project:

1. Competition of Uses shows that the most profitable use to generate rent can be achieved by building rehabilitation.

2. Economics of Succession shows that the existing improvments cannot be used and also the cost of demolition of the improvements and the cost of new building is not profitable.

3. Principle of Change shows that the technical, social and economic changes have altered the use of the building, the highest and the best use of the building is changed.

4. Regulatory Power of the Government requires to maintain and preserve the outer appearance of the buildings to blend in with other neighborhood structures.

Neighborhood life cycles generally pass through the Rehabilitation cycle. Preservation of neighborhood requires an active effort: Maintenance to counter aging and correction of the obsolescence by remodeling.

The steps in a typical building rehabilitation project is reviewed in Fig. 1 followed by a few case studies, so as to provide a feel towards developing a methodology for identification of systemic impacts of rehabilitation projects.

Stochastic Evaluation of Urban Renewal Alternatives

The evaluation of several alternative rehabilitation plans or projects including the alternative to do nothing to avoid further urban blight is considered a major step of the urban housing planning process. This evaluation requires an identification and establishment of a set of adequate criteria which can be used to select the "best" plan or project for implementation. The need to evaluate proposed alternatives and make decisions among them is one of the most pressing yet difficult requirements. When one considers the beneficial and disbeneficial impacts which built environmental systems can have -- for example, the development of access to employment opportunities, the movement of valuable resources to places where they can be utilized more effectively and, on the other side of the ledger, the creation of hazardous and unsafe impacts on the environment that can affect both the users and nonusers of the system -- feelings of uneasiness are difficult to overcome concerning the manner in which decisions affecting these important aspects of life might be made. One certainly would want to ensure that all possible avenues of approach had been explored so that the alternative providing the maximum net benefits for the users and those impacted, for a given load of required financial outlays would be both detected and chosen.

Many resources that were formally viewed as being unlimited are becoming scarce or at least reduced in supply. This has caused the thrust of present-day decision-making to be directed toward the allocation of resources. The implications of decision-making based on the allocation of resources has thus become of paramount importance to engineers working as planners, managers and decision making since engineers by warrant of their profession create resources at the expense of other resources as a means of contributing to a more desirable environment for human beings.

In the past, the major concern in project evaluation was centered only on how the created resources affected the prime societal resource, the monetary economy. The costs of the created resources were viewed in terms of the dollars they would cost to construct and the benefits of these same resources were viewed in terms of the dollars they would save. The ability to consider projects on this monetary scale allowed engineers to evaluate their projects with a number of methods based on the discipline of engineering economics, the rate of return, the benefit-cost ratio, incremental cost-benefit ratio being some of these techniques.

Engineers still rely on economic analysis as a means of evaluating project alternatives, although economics of integration needs to be better tied to economics of scale. The value of a rehabilitated building facility is premised on the dollar costs and the dollar value of the benefits of the facility. In essence this evaluation tests the impact of a facility on the economic resources of an area. The problem that confronts engineers is that the use of economic analysis methods for evaluating alternative projects, the assumption is made that the economic resource is the prime resource diminished. In order to evaluate the gained resources, such resources as mobility measured by lesser travel time, energy conservation must first be assigned dollar values in a subjective manner. However negative environmental impacts of congestion, air pollution, noise, increased traffic, land-use, drain on water supply, sewage and other utilities need to be also assessed. Only when this step has been taken can the dollar resources diminished by alternative facilities be compared to the dollar value assigned to the resources provided by the facility. This comparison via a number of possible analytical vehicles, yields an assessment of the net benefit of the project. Provided by this method is a means of representing by a single number (probability of maximum social/community benefit) the benefits and costs associated with alternative facilities. Admirable results if the initial assumption that the economic resource is the prime resource diminished is accepted by all concerned, could be obtained. The negative impacts of increased energy consumption, increased peak-hour congestion within narrower time periods; and the disintegration of a community and resulting blight would reduce the quality of life which, may not be amenable to monetary quantitative analysis in a satisfactory manner. In view of the above considerations, a comprehensive resources-environment-energy-economics-social basis is needed for functional planning, design and decision making. This proposed method is a step in this direction.

The base for stochastic resource based environmental analysis is chosen to be least social cost. Least social cost can be defined as what people will have to give up in terms of resources to gain the facility. The above paragraphs demonstrate that this base can be applied directly to environmental resources. However, with system resources, it becomes necessary to express "maximum social benefits" in terms of least social cost. As a consequence, statements concerning least social cost will vary depending on whether the element being modeled is an environmental element or a system element. This difference is largely one of what one would like the facility to do and what one would like the facility not to do. When talking about the facility's effects on the environment it can be said that one would like to "take" the least amount of resources. Defining resources in terms of supply and demand, what one would prefer to "take" would be something of abundance for which little desire exists.

The systems model, on the other hand, is the reverse of the above for here one would prefer to "take" something that is in little existence but for which there is a high demand. As an example, it may be important for a rehabilitation facility under study to interchange with other existing and/or planned facilities. As the existence of good building locations decrease it becomes more important to "hit" these locations.

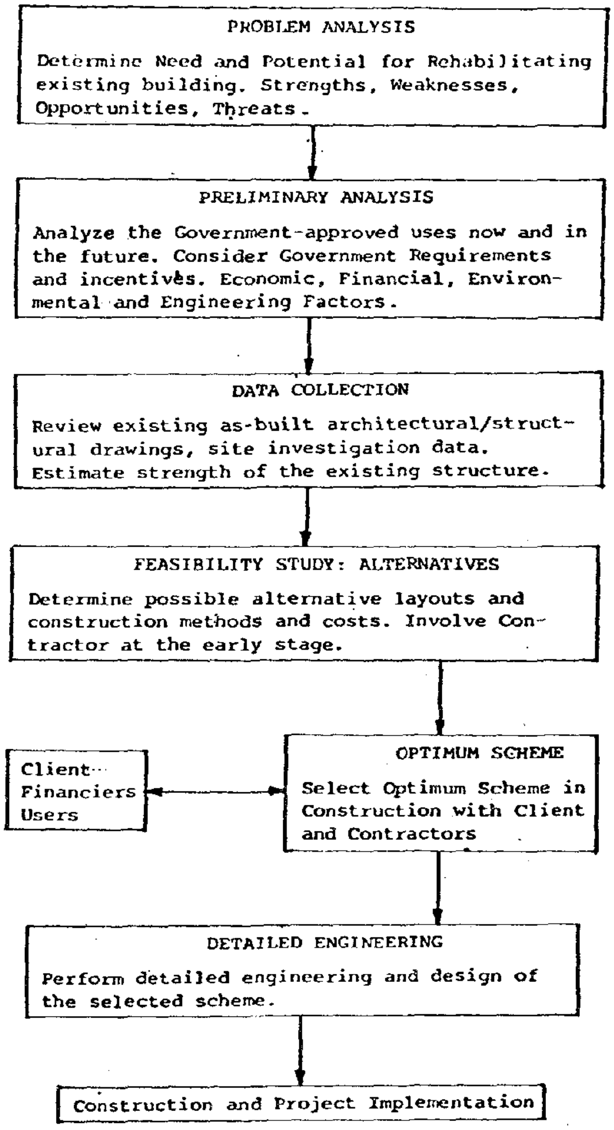


Fig.1: BUILDING REHABILITATION PROJECT STEPS

One of the alternatives is Do-nothing which increases urban blight as a function of time. The costs and economics of abandonment and rehabilitation need to be looked into very carefully, together with environmental effects.

Since housing is one of the most important factors in the quality of life, both individuals and governments are concerned that property owners maintain and rehabilitate their houses at optimum levels. Poor maintenance and premature abandonment of housing units generate excessive social costs.

Resource Categorization and Analysis

Resources can be divided into two groups: (a) Those relating to the housing system (system resources), and (b) those describing the environment (environmental resources). The basis of the modeling is to categorize both types of resources into easily discernable forms or levels of importance in order to establish supply and demand in a probabilistic format. For considerations of housing development projects involving rehabilitation, urban renewal, or do-nothing alternatives causing further urban blight. The following list represents the resources, as an example. This list is not comprehensive.

I. Housing System Resources: Built Environment
   a. Function
   b. Aesthetics
   c. Community attractiveness
   d. Cost
   e. Travel Time and Mobility
   f. Safety and Security
   g. Land Use Patterns, "Compact City" concept
   h. Urban Sprawl (short term, long term effects)
   i. Energy Conservation/Utility Costs
   j. Maintenance and Obsolescence

II. Environmental Resources
   a. Physical features
   b. Noise
   c. Replacement housing
   d. Crime Law Enforcement Problems
   e. Energy
   f. Ecology

The probability of supply of a resource category can be defined as the relative abundance of that category within its resource, the value of which is estimated as the ratio of the number of locations or areas where the category is found to exist to the total number of locations or areas where all categories pertaining to the resource are found to exist. The probability of demand of a resource category can be defined as the desire for that category within the boundaries of a study area, the value of which is determined subjectively and expressed as a percentage (i.e., 0% = no demand, to 100% = total demand), depending on the social, economic, and political characteristics of the area under study.

The supply and demand modeling approach provides the relative value of importance of each category within a given resource and, at the same time quantifies the importance of a resource on a spatial basis. With this modeling technique the desire is to identify the facility that will result in the least social cost to environmental resources and the maximum social benefits to the system resources. Since a proposed facility will diminish some categories of environmental resources, the least social cost location would be achieved if the resource categories so diminished are of low value. The best location for the building would be through an area where the environmental resource category is of very high supply and very low demand. Probability is used as a unifying integrating concept to deal with parameters of different disciplines.

The system resources are analyyzed by identifying the facility with maximum social benefits. This is accomplished by crossing areas where the resource category is an important system component. Maximum social benefit will occur where a system resource category exists with little supply but high demand. This suggested methodology needs to be further developed for rehabilitation and urban renewal projects.

CONCLUSIONS

A stochastic resource-environmental analysis method is suggested in this abridged paper. It represents an attempt to provide a technique applicable to the analysis of several alternatives concerning rehabilitation and urban renewal involving a large number of parameters. However, the key point is not the elaboration of this particular method but rather the demonstration of the need and benefit achieved from the comprehensive analysis of housing systemic (built environmental) and environmental impacts. Engineering, in general, and those in the building field specifically have been criticized severely for what is termed insensitivity to the needs of inner city cores caused by their "myopic" vision, especially in rehabilitation and urban renewal projects. Many projects are being blocked or delayed that could well prove highly beneficial based on economy of integration rather than mere economics of scale. A prime reason for this difficulty is the inability to demonstrate need in the face of conceptually larger environmental considerations. This situation will continue to grow until a means of total resource evaluation is achieved. Probability is used as an integrating tool to deal with parameters involving multiple disciplines: architecture, urban planning, urban economics, community development, energy conservation, transportation mobility, air pollution, Finance/Costs, Safety and Security, Politics to get the probability of maximum social benefit, to compare alternate projects.

Acknowledgements

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USE OF NON-DESTRUCTIVE METHODS TO EVALUATE AND INVESTIGATE CONDITION OF BUILDINGS AND BRIDGES

Gajanan M. Sabnis
Professor of Civil Engineering,
Howard University, Washington, D.C.; Also,
Vice President, Research and Development,
Sheladia Associates, Inc., Riverdale,
Maryland.

Leonid Millstein
Senior Research Engineer,
Sheladia Associates, Inc.,
Riverdale, Maryland.

SUMMARY

Use of non-destructive type testing during inspection of various components of buildings and bridges, enables one to determine their condition both qualitatively and quantitatively and to accumulate data for the history of the structure. Such inspection can indicate that physical testing may be required during it's lifetime for safety and proper serviceability. The application of non-destructive testing as presented in this paper cites the following types of examination of structure: (a) Acceptance criteria and control; (b) Preventative inspection; (c) Examination with non-destructive equipment; and (d) Inspection prior to rehabilitation. The procedures are discussed in detail to throw light on parameters such as geometry, strength, and deformation and other material properties as applicable to the strength and serviceability of the structure.

INTRODUCTION

Building and bridge inspections are a part of the management organization for structures and contribute to minimizing both the direct costs of maintenance and the consequential costs of load restrictions and diversions due to defective structures.

Visual examinations alone may not be adequate for the inspection of some modern structures of advanced design. The application to building and bridge inspection of research techniques for the investigation strength, elastic and non-elastic parameters, stressed state, corrosion, for detection of fatigue cracks in steel, etc., can be developed just using non-destructive methods.

Non-Destructive Testing Methods

Non-Destructive testing is the name given to all test methods which permit inspection of materials without impairing its usefulness. It's a science developed with emphasis on obtaining better physical measurements of material with quantitative information. Non-destructive testing falls into two main categories: Those which detect, identify and locate anomalies and those which correlate non-destructively measured quantities with properties that require some permanent change in the material, in order to be directly quantified.

At the Civil Engineering University in Moscow, USSR, a special team was organized consisting of highly educated professionals who were responsible for using and introducing all new non-destructive methods into inspection and evaluation of existing buildings and bridges. The main feature of that team was to provide a number of inspections to get information about "real" behavior of structures. There were a few types of inspections such as:

1. Principal inspection, which has to be provided at intervals not longer than six to seven years. A special kind of equipment has to be used to enable all parts of the structure to be investigated;
2. General inspection, which must be made intermediately at intervals not exceeding two to three years; and
3. Special inspection, if it is required.

The application of non-destructive testing cites relates to the following types of examination of structure:

Acceptance Criteria and Control

This should be carried out prior to the structure being put into service condition.

Preventive Inspection

This includes testing of components of structure to inspect their condition, which may change substantially due to environmental and other affecting factors. The results enables one to determine the nature and need of repair and rehabilitation and to eliminate hidden defects, if any.

Examination with Non-Destructive Equipment

This is performed to evaluate the causes of defects and their remedy.

Inspection Prior to Rehabilitation

This is performed in order to obtain qualitative data on actual conditions in the structure and to evaluate precisely the volume of work for reconstruction and rehabilitation.

Modern structural testing carried out with the instrumentation available today, may be divided into:

1. Structure strength check; and
2. Experimental investigations.

In both causes use of non-destructive methods for examination of existing structures is preferable.

Types of Non-Destructive Tests

The modern non-destructive test methods enable us to suggest the following classification:

1. Mechanical Non-Destructive Methods (NDM) (use power test of surface);
2. Acoustical NDM;
3. Vibration (oscillation) methods;
4. Radiography NDM (use penetrating nuclear radiation);
5. Magnetic NDM;
6. Electrical NDM:
7. Radio wave NDM; and
8. Other

Mechanical NDM may be further divided into:

a. Method of plastic print (impression);

b. Method of elastic restoration (scleroscopy); and

c. Method of small local failure.

Surface strength of material can be determined using these methods. To investigate strength inside structure mechanical NDM must be used together with extra methods, i.e., acoustical.

Acoustical NDM consists of:

a. Ultrasonic;

b. Acoustic emission; and

c. Impact wave method.

All of these methods could be used for investigation strength, physical mechanical properties of material and structure, acoustical parameters and homogeneous; i.e., ultrasonic examination provided a very effective three dimensional method for use in the internal evaluation of crack-type discontinuities, but rather poor for the evaluation of porosity.

Vibration (Oscillation) Methods

This method may be interpreted as:

a. Method of forced oscillation; and

b. Method of free oscillation.

These methods enable one to determine and investigate dynamic modulus of elasticity, dynamic modulus of shear, dynamic Poisson ratio, and logarithmic decrement.

Radiographic Method could be divided into:

a. Radioisotopic;

b. X-ray;

c. Neutron method.

Radiography is a very effective means of internal non-destructive examination. However, the results do not exactly parallel those obtained with ultrasonic, as radiography produces only a two dimensional image. These parameters may be determined by the above method: thickness of protective coating, size and location of bars in reinforced concrete, defects in weld, thickness of structures and moisture.

The magnetic method is primarily valuable in locating surface cracks and imperfections just below the surface of steel elements. Size, location and directions of bars also can be investigated.

Electrical - "Capacitance - Resistance" and inductive methods can be used for investigation of stressed state of structure in elastic and non-elastic stages of structures work.

The above classification of Non-Destructive Methods (NDM), which was suggested, is tentative. With the development of NDM it will be wider detailed best for the reflection of the situation of today; mutually complimenting each other, the NDM approach apparently can be adopted.

POSSIBILITIES OF NON-DESTRUCTIVE METHODS FOR INVESTIGATION OF SOME PROPERTIES AND CONDITION OF CONCRETE.

Properties to be controlled and condition of concrete	Non-Destructive Methods of Control							
	1	2	3	4	5	6	7	8
Strength	x	x	x					
Adhesion with bars in reinforced concrete		x	x			x		
Hardness	x							
Stressed state		x	x			x		x
Ultimate strain		x	x			x		
Elastic properties		x	x			x		
Damping		x	x					
Thermo-conductivity								x
Density				x		x	x	x
Porousness		x		x		x	x	x
Moisture				x		x	x	x
Large single internal defects		x		x			x	x
Small distributed defects of structure including micro-cracks		x	x	x		x		x
Roughness						x		

Analysis of the Table shows that not one of the methods is universal. Most widely used methods are acoustical, vibration, radiometric and electrical.

At the same time very important properties of concrete, i.e., such as deformation and adhesion with bars in reinforced concrete, do not have methods which can be used for their determination. Nevertheless it is posible to see from the comparison of the different methods that complex inculcation of those will enable one to estimate all important properties of concrete.

CONCLUSION

1. Non-Destructive Methods (NDM) provide information about the existing conditions and carrying capacities of the various components of structures and their variation with time. NDM also helps define more precisely the extent of the individual defects in the structure.

2. Non-Destructive Methods (NDM) enables one to determine the strength and deformation characteristics of structures. They also help define sections of reinforced concrete temperature-moisture conditions in the structural components of the buildings, openings, or cracks of the joints, and acoustical characteristics of the structures.

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INVESTIGATION TECHNIQUES OF MICROCRACKS FORMATION, PROPOGATION DUE TO APPLICATION OF COMPRESSIVE STRESS FIELDS

Kenneth Derucher
Civil Engineering
Stevens Institute of Technology
Hoboken, New Jersey 07030

SUMMARY

The major objective of this investigation was to directly observe the formation and/or propagation of microcracks in concrete both before and after application of compressive stress fields.

Concrete, under compression, fails when microcracks have propagated to the extent that the concrete will not support the applied loads. Many investigators have implied that the failure mechanism of concrete is related to internal microcracking. However, due to the limitations in the techniques employed, the detection of microcracks was somewhat uncertain.

The scanning electron microscrope (SEM) was chosen as the viewing apparatus because of its distinct advantages to directly observe the formation and/or propagation of microcracks.

Microcracks were found to exist in concrete prior to application of compressive stress fields in the form of shrinkage microcracks (initial bond microcracks). As the compressive stress field is increased, these microcracks wide and propagate until failure occurs.

INTRODUCTION

Many investigators have implied that the failure mechanism of concrete is associated with internal microcracking (1-6). The formation and propagation of such microcracks have been studied indirectly by sonic velocity, acoustic methods, and by the observation of macrocracks on the surface of the models. Robinson (7) and Hsu et al. (8) have directly observed the formation and propagation of microcracks by x-ray analysis. Due to the limitations in the technique employed, the detection of microcracks was somewhat uncertain.

In addition, Hsu et al. used a light microscope at 40X magnification to verify the results of the x-ray analysis. They examined cross sections of concrete (0.15 inches thick) both before and after application of compressive stress fields. In those concrete models which were examined after application of compressive stress fields, the concrete was sliced perpendicular to the direction of the applied load. Prior to slicing and examination of the concrete models, they were subjected to various compressive stress fields and the loads were subsequently removed.

According to Hsu et al., 3 types of microcracks were identified: bond, matrix, and aggregate microcracks. Further, bond microcracks (microcracks between the cement mortar matrix and aggregate particles) exist in the form of shrinkage microcracks prior to application of compressive stress fields. These initial microcracks begin to propagate at approximately 30-40% of the ultimate strength. The stress-strain curve deviates from linearity at this point, and there is an increase in the lateral expansion of the concrete. Matrix microcracks (microcracks in the cement mortar matrix) are formed by propagating bond microcracks at about 70-90% of the ultimate strength. Aggregate microcracks occur just before failure.

Hansen (9), in an attempt to verify the conclusions of Hsu et al., disagreed on some aspects of their findings. Hansen also tried to observe microcracks in concrete both before and after application of compressive stress fields, but the compressive stress fields were not removed prior to observation. Hansen applied a purely axial compressive stress field to the concrete models and observed (using a light microscope at 50X magnification) the formation and propagation of surface microcracks. Under a purely axial load, microcracks are believed to originate in the center of the concrete and propagate to the outer surface. Hansen, however, did not find bond microcracks in the form of shrinkage microcracks prior to application of compressive stress fields under magnifications as high as 1000X with the light microscope. Hansen discovered that bond microcracks (under a magnification of 50X) occurred at about 45% of the ultimate strength unlike the 30-40% figure found by Hsu et al. He agreed that matrix microcracks occur between 70 and 90% of the ultimate strength, and that aggregate microcracks occur just before failure.

It is apparent from the differences in the data received and the techniques employed by Hsu et al. and Hansen that further basic research is needed in the field of concrete microcracking.

A better understanding of the failure mechanism of concrete (at the microlevel) may provide a more knowledgeable understanding of the engineering properties of concrete, possibly leading to developments for improving these properties and subsequently improved use of concrete materials.

The major objective of this investigation was to observe directly the formation and/or propagation of microcracks in concrete both before and after the application of compressive stress fields. The compressive stress fields selected were 15, 45, and 75% of the ultimate strength of concrete.

It was not known what effect, if any, aggregate shape would have on the formation and/or propagation of microcracks in concrete. As a result, 2 distinct concrete models were used with the following parameters (Table 1).

Design of Experimental Investigation

It was decided to use the scanning electron microscope (SEM), model AMR 900 as the viewing apparatus (to observe microcracks in concrete directly) because of its unique capabilities and its distinct advantages over other viewing apparatus such as the light microscope and the transmission electron microscope. With the SEM it is possible to scan a 1 in^2 area, to magnify the same area 100,000X, and to obtain a relatively clear, sharp photograph.

Two SEM limitations were encountered in this type of investigation. First, there was only a 5" X 7" X 1-½" (12.7 cm. X 17.8 cm. X 3.8 cm.) usable space in the high-vacuum (10^{-6} torr) chamber. Since the major objective of this investigation was to observe microcracks directly in concrete under application of compressive stress fields, a loading apparatus (Fig. 1) had to be designed which would allow for the maximum size model possible (Fig. 2) and still be placed in the SEM vacuum-chamber in its entirety. The loading apparatus had to be designed to allow for the direct observation of microcracks when they form and/or propagate; this necessitated the eccentric loading of the concrete models. Thus, the size of the SEM vacuum-chamber controlled the size of the loading apparatus as well as the size of the concrete models.

The second limitation of the SEM was that in a high vacuum the concrete models must be relatively free of moisture; otherwise, due to the moisture in the concrete models, optimum operation cannot be reached. The models may contain molecular moisture; any moisture in excess of this amount cannot be tolerated. This limitation required developing a delicate 4-cycle drying process that would not include microcracking. The 4-cycle drying process consisted of: air drying, desiccator dyring (with silica gel as the desicant), vacuum-desiccator drying, and oven drying. In each cycle the models were dried to constant weight (as determined by daily weighing) before advancing to the next cycle. The entire process lasted approximately 25 days. To confirm the fact that the drying process would not include microcracking, a procedure was established with the use of the light microscope.

RESULTS

Shrinkage Microcracks

The results indicate the existence of shrinkage microcracks both in concrete containing rounded aggregate and in concrete containing angular aggregate. Neither the delicate 4-cycle drying process nor the SEM in obtaining a high-vacuum created, propagated, or widened these shrinkage microcracks. Many shrinkage microcracks were encountered in the concrete models, though only a few photographs will be shown. Fig. 3, obtained with the light microscope at an original magnification of 200X, is a typical shrinkage microcrack, in concrete containing rounded aggregate, prior to the application of the 4-cycle drying process. Similar shrinkage microcracks were encountered in concrete containing angular aggregate. These shrinkage microcracks are known as initial bond microcracks. It was concluded from Fig. 3 and similar photographs that microcracks exist in concrete prior to application of compressive stress fields.

Fig. 4, also obtained with the light microscope at an original magnification of 200X, is the same shrinkage microcrack shown in Fig. 3, but after the 4-cycle dyring process. In a comparison of Figs. 3 and 4, there do not appear to be any differences, nor was there any change in the existing microcracks or development of new ones. As a result, it was concluded that the 4-cycle dry-

Table 1 - Parameters Considered

Parameters	Model 1	Model 2
Ultimate Strength	3000 lbs/in^2 (211 kg/cm^2)	3000 lbs/in^2 (211 kg/cm^2)
Coarse Aggregate Shape	Rounded	Angular
Fine Aggregate Shape	Rounded	Angular
Top Size of Coarse Aggregate	0.50 in. (1.27 cm)	0.50 in. (1.27 cm)
Curing Time	28 days	28 days
Curing Temperature	70°F (21°C)	70°F (21°C)
Curing Relative Humidity	29 ± 2%	98 ± 2%
Number of Samples	4	4

ing process did not create, propagate, or widen shrinkage microcracks.

Microcracks in Concrete Containing Rounded Aggregate.

Figs. 5 through 8 are photographs of microcracks obtained with the SEM in concrete containing rounded aggregate both before and after application of compressive stress fields. Fig. 5 is a photograph of the scanning area of the concrete model prior to application of compressive stress fields: shrinkage microcracks are not obvious at this magnification. By magnifying and photographing the encircled area, Fig. 6 is obtained.

It is apparent from viewing Fig. 6 that shrinkage microcracks exist in concrete containing rounded aggregate. This shrinkage microcrack is not merely an initial bond microcrack (as described by Hsu et al.) but has matrix microcrack extensions. The average width of the microcrack in Fig. 6 is approximately 3 microns.

Applying a compressive stress field of 15% of the ultimate strength, a value within the straight line portion of the stress-strain curve, and viewing the encircled areas of Fig. 6 results in Fig. 7. It would appear that the microcracks have doubled in size (6 microns) and the matrix microcrack in the upper right hand corner of Fig. 6 has propagated.

By increasing the compressive stress field further to 45% of the ultimate strength and viewing the same area as in Figs. 6 and 7 results in Fig. 8. The microcracks have widened considerably, approximately 8 times (21 microns) their original width.

Microcracks in Concrete Containing Angular Aggregate.

Figs. 9 through 13 are photographs of microcracks, observed with the SEM, in concrete containing angular aggregate both before and after application of compressive stress fields. Fig. 9 is a photograph of the scanning area of the concrete model prior to application of compressive stress fields. Again shrinkage microcracks are not obvious at such a low magnification. However, if the encircled area is magnified and photographed, Fig. 10 results.

It is obvious, from viewing Fig. 10, that shrinkage microcracks exist in concrete containing angular aggregate. This shrinkage microcrack is not merely an initial bond microcrack (as described by Hsu et al.) but deviates from the aggregate-matrix interface (bond) into the matrix. The average width of the microcrack in Fig. 10 is approximately 2 microns.

Applying a compressive stress field of 15% of the ultimate strength and viewing the encircled area of Fig. 10 yields Fig. 11. The bond portion of the microcrack has increased in width approxomately 5 times and the matrix microcrack extensions have become much more pronounced.

Increasing the compressive stress field to 45% of the ultimate strength and viewing the same encircled area as in Figs. 10 and 11 results in Fig. 12. At this point, matrix microcracks begin to bridge bond microcracks with no noticeable increase in width, and the matrix microcracks become much more pronounced.

With a further increase of the compressive stress field to 75% of the ultimate strength, Fig. 13 results. There appears to have been a shifting of the aggregate particle and a widening of the microcrack.

DISCUSSION OF THE RESULTS

Shrinkage Microcracks

This investigation supports the results of Hsu, Slate, Sturman, and Winter in their study of shrinkage microcracks: microcracks exist in concrete prior to application of compressive stress fields. The results further show that shrinkage microcracks exist both in concrete containing rounded aggregate and in concrete containing angular aggregate. These shrinkage microcracks were not just initial bond microcrack extensions at right angles. Sturman (10) suggested that shrinkage microcracks may be formed by variety of processes, including carbonation shrinkage, hydration shrinkage, segregation due to settlement, and drying shrinkage. It is hypothesized that the shrinkage microcracks encountered in this investigation resulted from segregation due to settlement and, to some extent, hydration shrinkage.

Carbonation shrinkage occurs when any cement compound is stored in air and decomposed by carbon dioxide. The portland cement used in this investigation was of good quality and had just been manufactured and purchased. It had very little time to be exposed to air. Therefore, it does not seem likely that shrinkage microcracks were formed by carbonation shrinkage.

Hydration shrinkage occurs when the primary cement-paste volume decreases its volume during hydration, resulting in the formation of microcracks. This is said to be controlled by expansive cements. Since Type III-A cement (used in this investigation) is not an expansive cement, microcracks are possible.

According to Sturman, the influence of segregation due to settlement on the formation of microcracks may be analyzed by applying Stoke's law to the viscous material first formed when sand, cement, and water are mixed to form mortar. Stokes found that for very small solid particles suspended in a viscous fluid, the steady state or terminal velocity acquired by the larger and denser particles is greater than that of the smaller, less dense particles. Thus, in the sand-cement-water mixture the large sand particles will settle first, the fines next, and the extremely fine, flocculated particles last. This leads to a condition in which there is, adjacent to the aggregate, a thin film of fluid with an extremely high water to solids ratio. Eventually this water is absorbed by the adjoining cement paste which hydrates continuously and a thin space is left at this point on the aggregate. When this sedimentation occurs at the exposed horizontal surface of freshly poured concrete, it is referred to as bleeding. This phenomenon is likely the cause of shrinkage microcracks in concrete and is probably the major cause of shrinkage microcrakcs in this investigation.

The final possibility is drying shrinkage. Drying shrinkage occurs in plastic concrete if the rate of evaporation exceeds 0.1 lbs/ft.^{2} of surface area/hr. In other words, hydrostatic tension is present, resulting in shrinkage microcracks. If the concrete were cured in a water-saturated atmosphere, as it was in this investigation, shrinkage microcracks do not develop. Therefore, this possibility was ruled out.

It was not known why Hansen did not encounter shrinkage microcracks, because there was no information available as to the concrete materials and curing procedures used.

Concrete Containing Rounded Aggregate and Concrete Containing Angular Aggregate.

Hsu et al. stated that bond microcracks (which exist prior to the application of compressive stress fields) propagated at 30-40% of the ultimate strength. Hansen found that bond microcracks did not originate until 45% of the ultimate strength. In this investigation, bond microcracks (which exist prior to the application of compressive stress fields) did not propagate at all but merely widened under increasing compressive stress fields. The propagation of microcracks occurred only to the matrix extensions.

Both Hsu et al. and Hansen further agree that matrix microcracks occurred between 70 and 90% of the ultimate strength. In this investigation, matrix microcracks were extensions of bond microcracks and existed prior to application of compressive stress fields. These matrix microcracks were at right angles to the bond microcracks. Under increasing compressive stress fields (as low as 15% of the ultimate strength) matrix microcracks widen and propagage to the point that they begin to bridge bond microcracks.

This investigation further shows that at 45% of the ultimate strength the bridging of bond microcracks is about completed. At 75% of the ultimate strength the matrix microcracks start to bridge one another.

The differences between the results obtained in this investigation and those of previous investigations are easily explained. The depth of field and scanning ability are the 2 features that make the SEM particularly well suited for fractography. Its depth of field is many times greater than that provided by the light microscope (such as that used by Hsu et al. and Hansen) for equivalent magnifications. This feature permits both the peaks and valleys normally encountered on rough fractured surfaces to be imaged in focus, even at relatively high magnifications. The light microscope used in previous investigations had poor depth of field at 40X and 50X magnification. This poor depth made it difficult to distinguish between actual microcracks in the matrix and very porous mortar. Even the use of the light microscope in this investigation presented doubt as to the existence of an actual microcrack. At lower compressive stress fields Hsu et al. and Hansen may have mistaken the microcracks to be very porous mortar (since the microcracks were only 3 or 4 microns wide) and only when the microcracks reached a proportional size could a difference be made. This may account in part for the higher values received by Hsu et al. and Hansen in their investigation of matrix microcracks.

In addition, Hsu et al. removed the compressive stress fields prior to observation of microcracks. Many microcracks which formed may have gone unnoticed, because they may have closed due to the relief in stress. Again, this may account for in part the high values received by Hsu et al.

In Hansen's investigation, surface microcracks were observed. Hansen applied a purely axial compressive stress field to the concrete models. Under an axial load microcracks are believed to originate in the center of the concrete models and then propagate to the outer surface. Under this assumption, microcracks would not be noticed on the surface of the models until higher compressive stress fields were reached. This may account for the differences received by Hsu et al. and Hansen as far as bond microcracks are concerned.

CONCLUSIONS

The following are some of the conclusions drawn from this investigation:

1. Procedures, techniques, and apparatus were developed and/or modified for the study of concrete fracture utilizing the SEM.

2. Microcracks were found to exist in concrete prior to application of compressive stress fields. These initial microcracks are shrinkage microcracks. These shrinkage microcracks were in the form of bond microcracks (microcracks at the interface between the aggregate and the matrix) with matrix microcrack extensions (microcracks in the paste).

3. Shrinkage microcracks (initial bond microcracks) propagate into the matrix and widen under an increasing compressive stress field. As the compressive stress field approaches the ultimage strength of concrete these microcracks become macrocracks and with time fail the member.

4. Under increasing compressive stress fields (as low as 15% of the ultimate strength) matrix microcracks widen and propagate to the point they begin to bridge bond microcracks.

5. At 45% of the ultimate strength of concrete the bridging of the bond microcracks is about complete.

6. At 75% of the ultimate strength of concrete the bridging of the matrix microcracks begins and as the compressive stress field is further increased it is conjectured that these microcracks will continue to widen and propagate until failure occurs.

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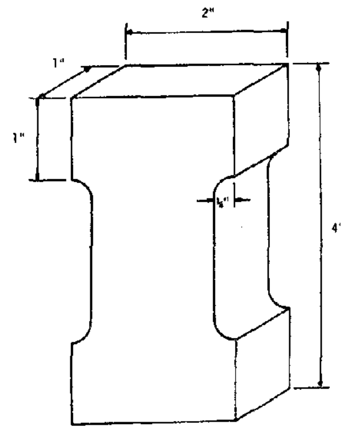

Fig. 2 Concrete Shape and Size (Model)

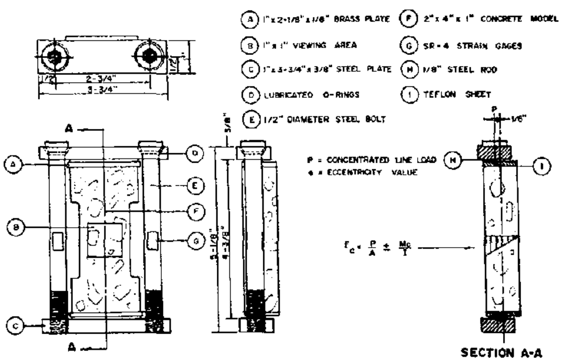

Fig. 1 Loading Apparatus

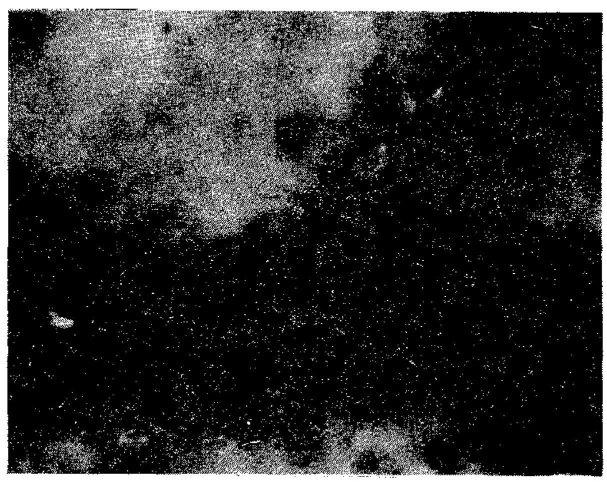

Fig. 3 Shrinkage Microcrack Prior to Application of Four Cycle Drying Process

Fig. 4 Shrinkage Microcrack After Application of Four Cycle Drying Process

Fig. 5 Scanning Area of the Concrete Model Containing Rounded Aggregate

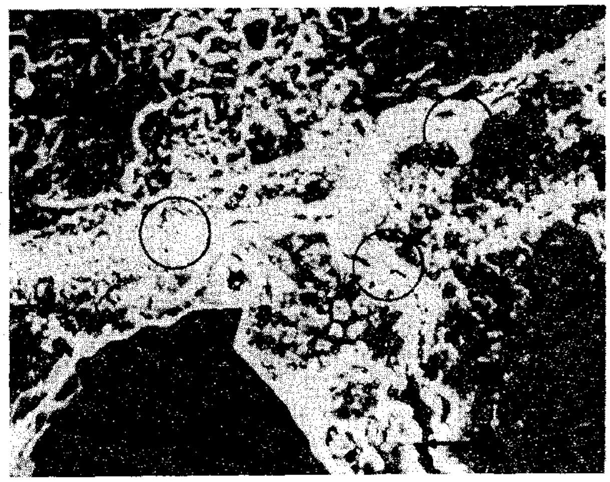

Fig. 6 Scanning Area Shown in Fig. 5 Magnified to Illustrate Existence of Shrinkage Microcracks in Concrete Containing Rounded Aggregate

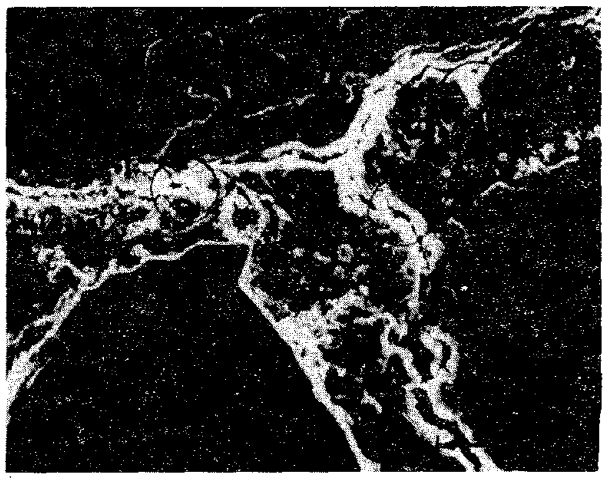

Fig. 7 Microcracks in Concrete Containing Rounded Aggregate Under Application of a Compressive Stress Field of 15 percent of the Ultimate Strength

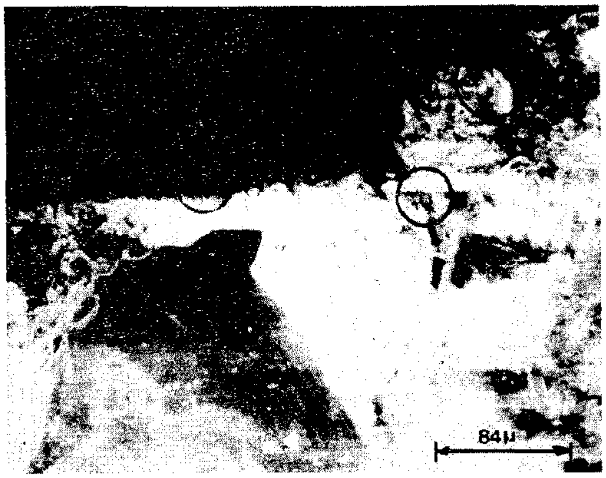

Fig. 8 Microcracks in Concrete Containing Rounded Aggregate Under Application of a Compressive Stress Field of 45 percent of the Ultimate Strength

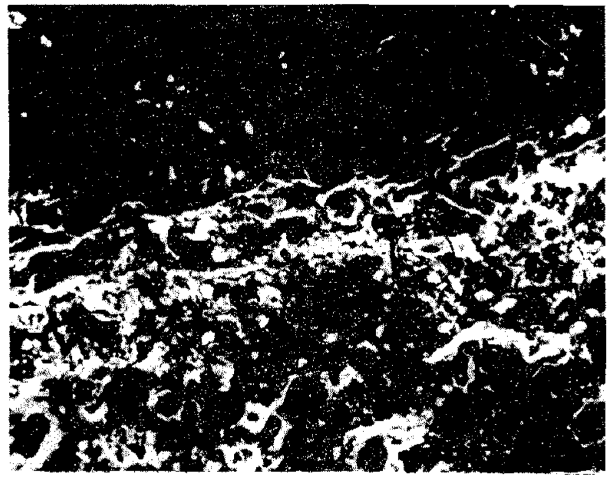

Fig. 10 Scanning Area Shown in Fig. 9 Magnified to Illustrate Existence of Shrinkage Microcracks in Concrete Containing Angular Aggregate

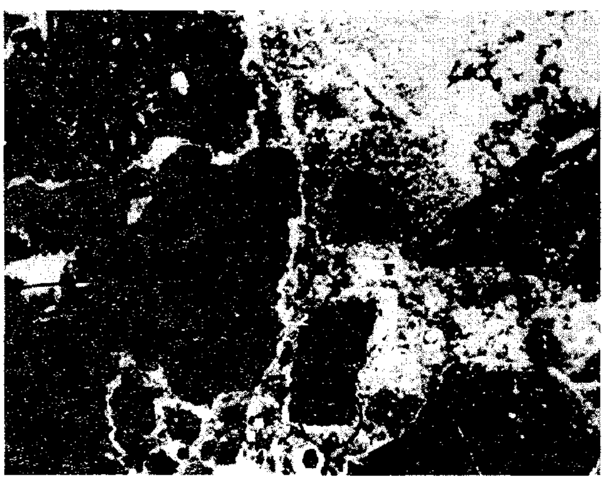

Fig. 9 Scanning Area of the Concrete Model Containing Angular Aggregate

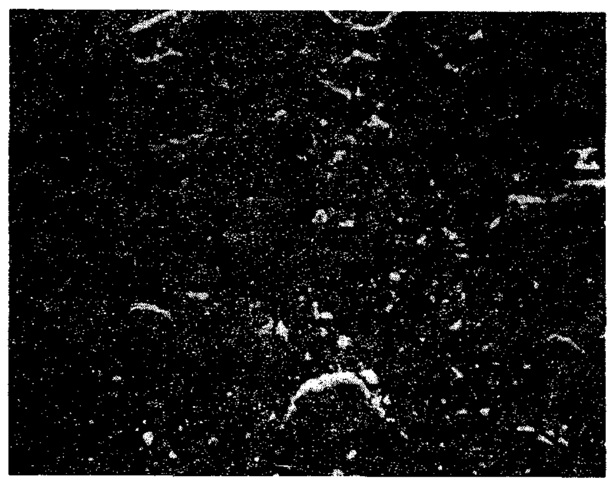

Fig. 11 Microcracks in Concrete Containing Angular Aggregate Under Application of a Compressive Stress Field of 15 percent of the Ultimate Strength

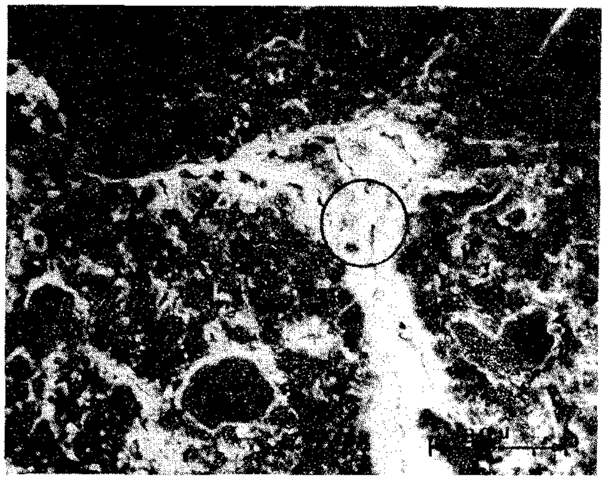

Fig. 12 Microcracks in Concrete Containing Angular Aggregate Under Application of a Compressive Stress Field of 45 percent of the Ultimate Strength

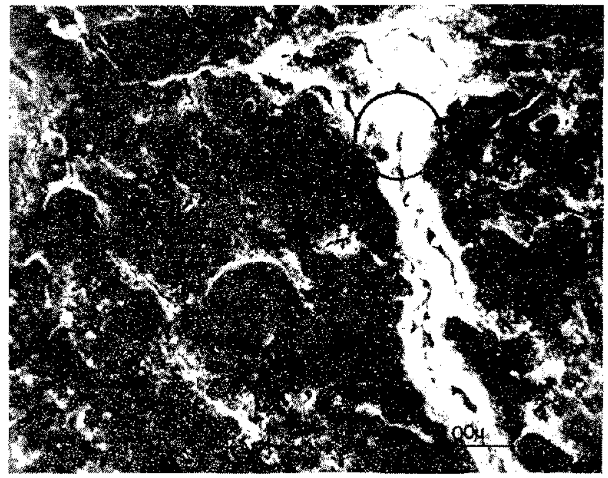

Fig. 13 Microcracks in Concrete Containing Angular Aggregate Under Application of a Compressive Stress Field of 75 percent of the Ultimate Strength

RESTORATION OF STRUCTURES WITH COMPACTION GROUTING

Dr. Chandra S. Brahma,
Professor of Civil Engineering
California State University, Fresno
Fresno, California
Consultant Sverdrup & Parcel and Associates, Inc.
St. Louis, Missouri

SUMMARY

Some of the typical problems associated with tunneling in soft ground and several mechanisms responsible for causing such difficulties have been identified. Various sources of ground movement and methods of effective ground control in tunneling, in general, and restoration in structures with compaction grouting technique, in particular, have been briefly discussed with an illustrative case history. The dynamic application of compaction grouting technique to simultaneous soft-ground tunneling operations eliminated or greatly reduced vertical as well as lateral movements that usually occur on the ground surface one or more tunnel diameters away from the center line.

INTRODUCTION

Experience has demonstrated that the construction of every soft ground tunnel is associated with a certain minimum loss of ground that cannot be prevented. The magnitude and character of the lost ground invariably depend on the nature as well as details of stratification of the surrounding soil, groundwater conditions, overall construction procedures and sequences, the details of tunneling methods and equipment, and the degree of care exercised during construction. Of several problems in tunneling in urban areas, the damage to adjacent or overlying structures and/or utilities due to ground movements around soft ground tunnels is probably the most critical one insofar as types of construction suitable for a particular site are concerned. Many of the design as well as construction considerations on a soft ground tunneling project would and should therefore be directed toward preventing excessive damage to structures near the tunnel. While available information relating soil and constrction conditions to ground movements, and ground movements to the distortion and damage of the structures are, at present, not sufficient to render reliable estimates of ground movement or damage potential above tunnels, decisions must often be made regarding the need for underpinning or protection measures, if any, and the cost of the work therewith during the initial phase of the project.

Unlike the more traditional approaches with structural support methods such as underpinning, contingency support with control jack piers or column/wall pick up, etc. which are by no means a cureall despite substantial direct and intangible costs associated therewith, the compaction grouting technique under certain conditions may offer an attractive alternative in the context of structural support and protection. The grout used with such technique is a stiff mortar, which is injected under pressure at the desired location, in order to form a bulb of solidified material distinct from the soil with accompanying displacement and densification of the surrounding soil. Although the use of compaction grouting is frequently restricted to the correction of loose soil conditions or raising of damaged structures, the dynamic application of such technique to simultaneous soft ground tunneling operations is a relatively new procedure. The primary objectives of this paper are, therefore, to briefly discuss behavior of residual soils or very compact granular soils during tunneling, to identify various sources of soil displacements about tunnels, to outline the compaction grouting program as implemented in the field with a recently completed tunneling project, and to present typical ground movement data obtained in a geotechnical instrumentation program carried out with the project.

TYPICAL PROBLEMS IN SOFT GROUND TUNNELING

The excavation of a tunnel in a stressed medium by any method invariably causes not only a substantial reduction in radial soil stresses near the tunnel but a significant increase in tangential stresses with consequent increase in shear stresses as well. As the circumferential stress approaches the ground's unconfined compression strength, the ground may stand initially and then start to ravel after a period of time (Brahma, 1976, 1977). Blocks and wedges of cohesive granular soil (transported or residual) lying above the tunnel or in the upper part of the working face tend to spall into the heading prior to installing appropriate support systems. The reason for the delayed failure with ravelling may largely be attributed to the increase in ground stress about the tunnel with time due either to adjustment of pore pressures, if any, or to stress concentration around discontinuity planes. While a low cohesive strength and/or adversely oriented planes of weakness can cause several spalls simultaneously, the ravelling phenomenon is likely to propagate by successive and concurrent disintegration leading ultimately to open cavities above the tunnel or even to sinkholes at the ground surface.

Should the tangential stress much exceed the ground's unconfined compressive strength, the ground is likely to fail almost immediately upon exposure due to short stand-up time. Granular soils run from unsupported face crown, and wall and the severity of such instability increases with more uniform gradation and decreasing density and cohesion or cementation.

Most severe stability problems and construction difficulties relating to high infiltrations, soil erosion, unexpected flows, and pore water pressures are associated in one way or other with the presence of water. Should seepage pressure toward the working face be permitted to develop in what would otherwise be fast ravelling or running ground, the soil may plausibly be transferred into flowing ground and advance like a thick liquid into the heading. A high hydraulic gradient can potentially increase the tendency to ravel or erode materials at the heading transforming slow ravelling ground into fast ravelling ground and if not controlled, such actions often lead to formation of sinkholes at the ground surface. Where an interface of permeable granular soil over relatively impermeable fine-grained soil is present in the heading, it is possible to dam or perch water behind the less permeable zones developing much higher water pressures than would otherwise be anticipated and to encounter high rates of flow as well as high volumes upon intersecting the more permeable zones. Any attempt made to dewater the overlying granular materials would more often than not fail to dry up such an interface. Consequently, the difficulties with seepage as well as erosion control along the interface are frequently fraught with uncertainties (Brahma, 1978).

GROUND MOVEMENT

Soil moves continuously toward the tunnel opening and contributes to the transverse settlement profile at ground surface. Of the various sources contributing to ground subsidence, ground losses from the face, shield, and tail void may be considered important. Soil migration with seepage and elastic or elasto-plastic strains as well as displacements due to the change of soil stress conditions about the tunnel, can give rise to further ground movements.

The volume of soil excavated at the tunnel face in excess of volume of the shield is ordinarily known as face loss. Running, ravelling, or flowing grounds frequently encounter severe ground losses through the face. Inadequate jacking pressure on a cutting wheel or removal of boulders manually ahead of the face can also contribute to face losses. Face losses with appropriate tunneling methods may be controlled by breast boards with soldiers or face jacks, closer plates or diaphragm gates, conventional grouting or other measures. While these means of face stabilization may result in satisfactory tunnels in moderate to slow ravelling grounds, mandatory use of compressed air or predrainage is frequently resorted to in grounds with adverse seepage pressures.

Plowing as well as yawing of the shield and overcutting caused either by the use of external devices such as beads, teeth, and poling plates or by advancing a straight cylindrical shield along a curved alignment result in loss of ground. If a shield is pitched upward or downward at an inclination other than that dictated by the path of shield movement in order to maintain specified tunnel grades, the excavated elliptical cross-section results in loss of ground due to over-excavation at the face as the ground compresses or squeezes below the shield and is displaced from above the shield. Loss of ground in a similar fashion occurs due to yawing when the shield is allowed to move irregularly from side to side. While shields fitted often with a bead to provide an over-cut permitting axial adjustments create a space outside the shield skin for grounds to subsequently cave in, the poling plates on the top half and the teeth projecting on the lower half of the perimeter of the shield are likely to produce further voids. Should the shield negotiate tight curves without becoming iron bound, additional loss of ground results from overcutting due to curvature of the alignment. An articulated shield with a relatively thin bead may not only facilitate steering difficulties with the curve, but reduce loss of ground as well. Grouting through the shield, although limited in its effectiveness, may be undertaken to fill some of the voids created by external devices.

Tunnel liners are frequently required to be erected under the protection of a shield tail with certain clearance. As the shield moves forward, ravelling may begin, for example, in the roof of the tunnel, allowing soils to move into the void left by the tail skin and clearance. Articulated shields requiring thin tail skin and relatively small clearance may somewhat reduce the loss of ground because of decrease in volume of the void. However, the annular void must positively be filled in order to prevent loss of ground. While the filling may be accomplished by expanding the liner out against the ground or by injecting materials into the void, use of air pressure or predrainage, if resorted to, may stabilize soils subjected to fast ravelling or running with consequent reduction in loss of ground. Grout filling near the shield may be performed under controlled pressure during each limited shove with appropriate seals between liners and shield tail to prevent forward flowing of backfill grout into the working chamber. Several considerations of the ground movement have been published elsewhere (Brahma, 1978; Hansmire and Cording, 1972).

The foregoing discussions outlined several sources contributing to ground losses in single tunnels. The magnitude of surface settlement is dictated by the volume of settlement trough at the surface and the lateral distribution thereof. Maximum settlement at only a little distance above the tunnel crown represents the immediate response of the ground to the intrusion facility offered by the presence of the tunnel opening. Ground settlements at shallower depths, more remote from the source, not only are more delayed in time but also are less in magnitude, as the volume of lost ground into the tunnel becomes spread out during upward migration from its source. While the mechanism with the propagation of lost ground about the tunnel has recently been dealt with elsewhere (Brahma, 1979) and will not be discussed herein further, the surface settlement profiles are often approximated by the Gaussian error functions with several empirical procedures (Peck, 1969; Schmidt, 1969) as an expedient to compare field measurements. With such procedures, the ground loss is assumed to be evenly distributed over the width of the tunnel and the width of the settlement trough is influenced by the ratio of the depth of cover to the diameter of the opening, the volume of lost ground, and geologic conditions.

CASE HISTORY: BOLTON HILL TUNNELS, BALTIMORE, MARYLAND

This section of transit system consists of twin tunnels, each approximately 18 feet (5.5 m) inside diameter and 21 feet (6.4 m) to 28 feet (8.5 m) apart. The twin tunnels connect three downtown subway stations and each is about 5500 feet (1676 m) long. The transit alignment includes horizontal curves of the order of 800 feet (244 m).

A total of 263 buildings are located within the zone of influence which is most susceptible to ground movements resulting from construction operations. While some of these buildings are scheduled for demolition, gutted by fire and in poor conditions, or already abandoned, a large majority of the buildings, which are two and three-story row houses with brownstone basements, masonry bearing walls, and wood floors and roofs, would not either be damaged substantially to warrant expensive underpinning effort or justify the cost of support even if the damage is widespread. Only forty buildings are deemed to to require protection due to their location, size, age, and function. These buildings have been scheduled for protection against damage resulting from ground subsidence-related tunneling operations by compaction grouting technique.

Subsurface Conditions:

The selected vertical alignment of the Bolton Hill line placed crown at a depth of about 40 feet (12.2 m) to 80 feet (24.4 m) below the ground surface. Miscellaneous man-made fill of 10 feet (3 m) to 15 feet (4.6 m) in thickness overlies the uppermost stratum of the Cretaceous age Patuxent formation. The deposit is mostly granular, consisting of silty fine to coarse sand, gravelly sand, and sandy gravel, and interbedded randomly with lenses of silt, clay, or both. Standard penetration resistance ranges from 6 to greater than 100 blows per foot, with the lower value generally occurring near the surface. A stiff to hard gray silty clay and clayey silt layer occurs within the deposit with plastic limit and natural moisture content coinciding in the range of 15 to 20 percent. Underlying the Cretaceous sediment are residual soils formed by a process of weathering and decomposition of the parent rock formation. The residual materials exhibit various degrees of weathering, ranging from completely decomposed materials with no indication of their parent rock to less decomposed materials which not only contain partially weathered and/or fresh rock components and remanent rock structures, but ordinarily retain some cohesion of the parent rock as well. Throughout most of the alignment, the tunneling is carried out in predominately granular soils with invert and lower face of the tunnel passing through residual soils. However, there are portions of the alignment where residual soils are encountered in the upper face extending to levels above the crown. The groundwater level along the alignment lies about 5 feet (1.5 m) below to 20 feet (6 m) above the crown of the tunnel.

Tunnel Construction:

Twin tunnels with prefabricated steel liners were driven in a southerly direction from the Southern Access Shaft at the Bolton Hill Station and completed first, followed by those in the northernly direction. Both tunnels were driven under compressed air with pressures less than 12 psi except for the initial approximately 280 feet (85.3m) of tunnel which were driven in free air with predrainage. In order to keep ground movements to a minimum, tunneling operations were carefully designed. Breasting of the face, use of articulated shield and limited bead size (9.5 mm maximum), positive filling of voids left outside the shield, maximum size of annular space between the tail and the liners (38 mm), and proportioning of the hood were stipulated. In order to minimize the length of unsupported soil over the tail void, grouting of an incompressible filler with an appropriate strength was used with every two feet of shove. The tunnel face from springline to the crown were fully breasted during tunneling operations within fifty feet (15 m) horizontally each side of forty buildings to be protected.

While the foregoing measures were able to prevent, or significantly reduce ground losses at the heading, the program of compaction grouting was undertaken in order to further compensate for such losses by recompacting the soils loosened by tunneling operations within the vicinity of the tunnel. Not only did such grouting affect a large zone below the existing foundation of the selected buildings, but settlements as well as lateral movements associated therewith were so reduced that these buildings were restored with minor repair without resorting to costly underpinning.

Compaction Grouting Program:

Of the forty buildings to be protected by compaction grouting, eight buildings were close to or actually above at least one of the tunnel routes. The remaining buildings were set back from the actual tunnel line. For the buildings, which were offset from the tunnel line, the grout pipes were placed about 5 feet (1.5 m) off the center line toward the building on 10 foot (3 m) centers. Where the tunnel went under a building, two rows of grout pipes were placed over the tunnel, each at 10 foot (3 m) centers and offset 5 feet (1.5 m) from the center line in opposite direction in staggered fashion. Grout pipes extended about 40 feet (12.2 m) beyond the projection of the affected building in the direction of the tunnel axis.

A stiff viscous mortar grout, consisting of well-graded sandy soil and Portland cement, with cone-slump consistency of about 2 inches (50.8 mm), was placed through vertical and inclined pipes 10 feet (3 m) above the tunnel crown in order to prevent excessive loading on the tunnel lining and the grouting operations were continued until injection pressures reached 2758 to 3447 kilo Pascal without sand blockage. Grouting operations with each grout pipe were begun sometime after the tail had passed below the end of the pipe and before movements were observed on the ground surface at that location. While the grout should only be as strong as the surrounding soil, considerable in place strengths frequently resulted from water losses during pumping and associated densifying effect. In order to be able to start injection process, the pipe was pulled out above 5 feet (1.5 m) at the beginning of or just prior to grouting in a particular pipe. The size of the initial void space below the pipe was increased from time to time by air blowing out additional soils in order to facilitate grout movement under the available pressure.

A second stage of compaction grouting was stipulated after the shield had passed a particular building, if the ground movements in the area so dictate. The second stage grout bulbs were to be placed a few feet above the primary grout bulbs prior to restarting the injection process. Actual average compaction grout takes were about 10 cubic feet (.2832 m^3) per lineal foot (.3048 m) of tunnel which is about six times the settlement trough volume anticipated with the ungrouted portion of the tunnel routes.

Monitoring Program and Results:

Construction performance during compaction grouting as well as tunneling operations was monitored. The monitoring program consisted, among others, of surface settlement points, deep settlement borris points, inclino-settlement columns, building settlement points, and piezometers. Intensive ground movement monitoring was concentrated in several test sections enabling a diagnosis of the seat of ground losses through both grouted and ungrouted portions of the tunnel alignment and the causes of loss of ground.

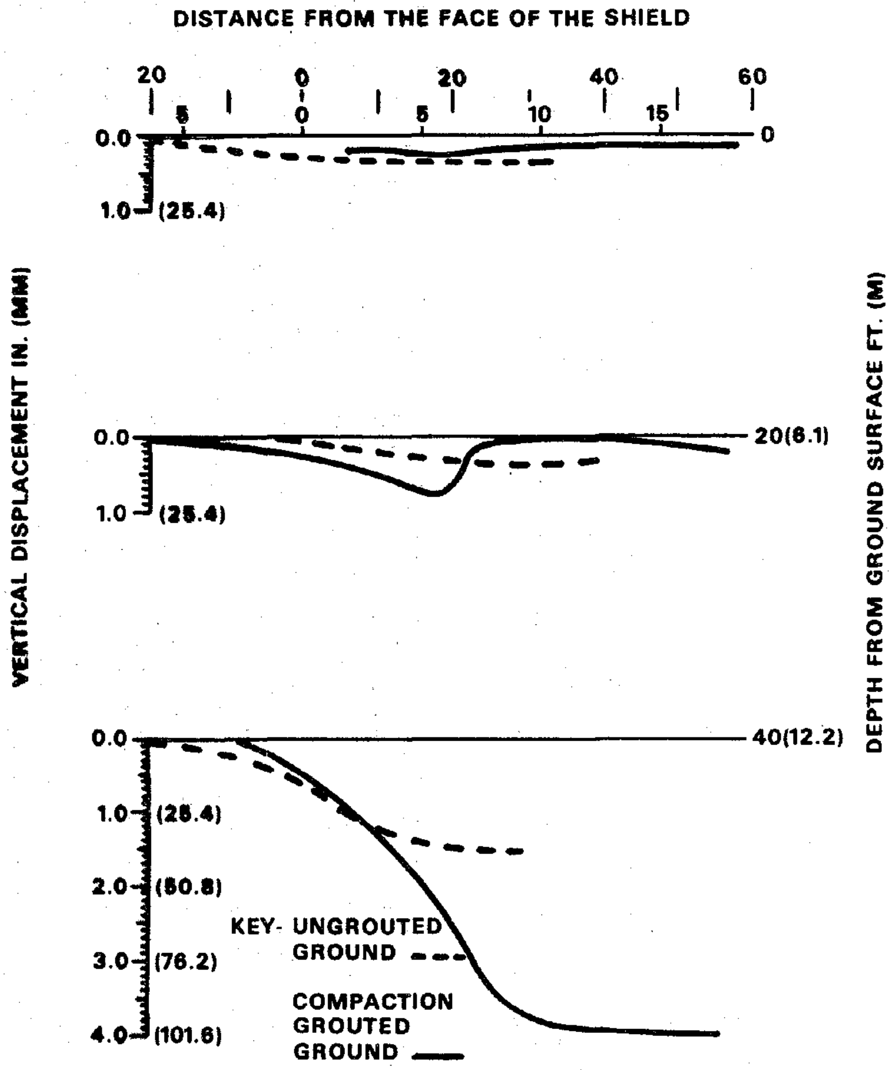

FIG. 1

VERTICAL DISPLACEMENT ABOVE THE THE CROWN OF THE TUNNEL (I.B.)

Figure 1 shows typical vertical displacements above the crown of inbound tunnel in grouted and ungrouted soils with depths below the ground surface. The deep settlement curves show that very little settlement occurred ahead of the tunnel face and that overexcavations due to either the plowing and yawing of shield or the use of bead at the leading edge of the shield and the inadequate filling of tail voids may be responsible for such lost grounds. In addition, the major portion of the settlement in ungrouted sections occurs from the tail of the shield to a point a few feet behind the tail just as the shield tail passes a particular station. Of more significance, however, is the fact that with compaction grouting, vertical displacements above the grout bulb are reversed and greatly increased below the grout zone.

Typical surface settlement troughs for grouted as well as ungrouted sections of the twin tunnels are shown in Figure 2. It can be observed that the total surface settlements in ungrouted sections developed a large,

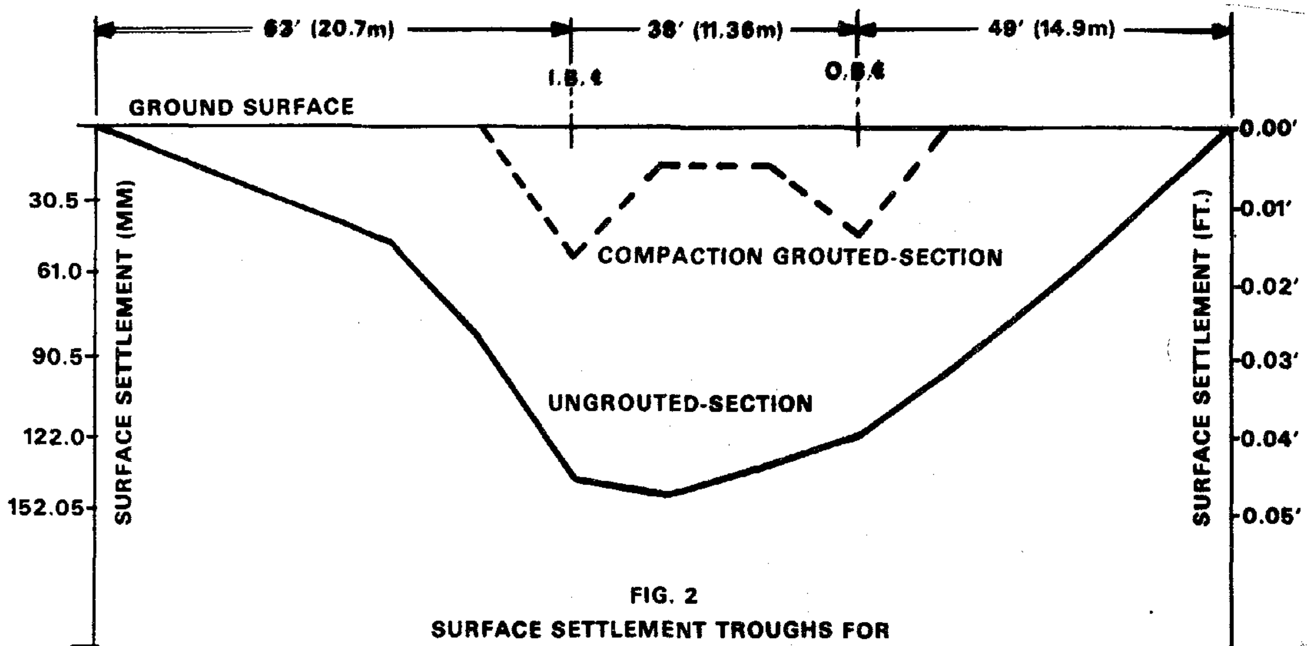

FIG. 2

SURFACE SETTLEMENT TROUGHS FOR GROUTED AND UNGROUTED SECTIONS

broad settlement profile with a settlement and volume of surface trough of about 0.6 inch (15.2 mm) and 3.53 cubic feet (0.1 cu m) per linear foot (0.3 m) of the tunnel respectively. While the magnitudes of settlement are small, the final settlement trough is more or less symmetrical about the midpoint between the centers of twin tunnels. The reasons for such broadness of the settlement trough may largely be attributed to loss of air resulting in development of minor seeps at the heading, especially where the Cretaceous deposits above the tunnel crown are known to be interbedded with a layer of stiff to hard silty clay and lenses of silt or clay. However, both settlement and volume of surface settlement trough in grouted sections are even smaller in comparison. A maximum settlement of 0.2 inch (5.1 mm) and volume of surface settlement trough of about 0.33 cubic foot per linear foot (0.03 m^3/m) of tunnel were recorded indicating that the compaction grouting program was successful in densifying materials in the pillar so as to cause minimal interference between the inbound and outbound settlement troughs.

The maximum settlement in excess of 4 inches (10 cm) were measured at a depth of 40 feet (12.2 m) below the ground surface probably due to the effect of grouting at high pressures. However, the settlements were progressively reduced near the ground surface. The ground movement underneath the buildings scheduled to be protected with compaction grouting technique was generally less than 0.25 inch (6.4 mm) requiring only minor repairs for their restoration. While good workmanship and strict adherence to construction procedures outlined previously may be responsible for modest settlements evicenced at this site, the attenuation of settlements near ground surface must have been influenced by the soil movements resulting from compaction grouting at high pressures and the volume expansion of soil in a limited region above the grout zone.

CONCLUSIONS

Compaction grouting operations, if carried out in the vicinity of the tunnel crown, has proven effective in protecting existing structures nearby against damage by significantly reducing vertical ground displacements and lateral soil movements associated therewith. Since such technique is not generally intended to compensate for large ground losses at the headings, it is imperative that proper construction procedures and good workmanship must be followed during tunneling operations. Furthermore, various schemes with compaction grouting involving detailed and close monitoring of ground movements should be carefully executed.

While there were no structures where underpinning was the clear choice of protection with the information at hand, it was believed that compaction grouting techniques might control anticipated settlements in forty buildings with less cost and inconvenience than other methods. Not only would the application of compaction grouting operations during tunneling reduce the ground movements over a large area, but the method has the ability to raise heavy foundations and light floor slabs simultaneously and under control should the field conditions so dictate. This technique has frequently been used as a method of repair after settlement has taken place in those cases where the probability of damage is small but a few buildings are likely to be damaged. However, for substantial and valuable buildings, generally with specific points of loading, such as heavily loaded columns, the cost of underpinning as related to the benefits might be much more attractive.

The compaction grouting operations to reduce surface subsidence due to tunneling on the present project appeared to be of a less critical nature than might otherwise be anticipated. However, the groting operations must be flexible but consistent with important economic advantages.

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WOOD AS CONSTRUCTION MATERIAL FOR REHABILITATION OF BUILDINGS AND BRIDGES

Dr. S. K. Malhotra,
Professor of Civil Engineering
Technical University of Nova Scotia
Halifax, Nova Scotia, Canada

SUMMARY:

Various factors affecting the serviceability of wood structures are discussed in the paper. Role and significance of some recent research and developments in wood engineering and structural wood design are given, as they relate to improved capacities of structures.

Inspection, investigation and evaluation of wood structures are mentioned in the paper. Good engineering practices, design and construction are briefly mentioned. Some techniques of upgrading of existing timber structures are given. Examples of evaluations, investigations and rehabilitations of timber buildings and bridges are outlined. Highlights are given of an investigation and a full-scale test on laminated segmental latticed roof trusses of 60 feet (18.29 m) built up by joining small dimension lumber with nails. Wood, a renewable resource and one of the most energy-efficient construction materials, can play a vital role in the rehabilitation of structures.

INTRODUCTION:

The demand for rehabilitation (reclaiming, recycling, restoration, renovation or simply repair) of old structures for adaptive or continuing use, is growing quickly and has become an important element in the design professions and construction industry. The reasons for rehabilitation of existing structures can be many - soci-economic, technical, aesthetic, historic, etc.

The process of rehabilitation of structures consists of multitude of components. As an integral part in this process, it is essential to carry out a detailed structural and materials evaluation. Before this evaluation is conducted on an existing structure, it is imperative to have a comprehensive understanding of the quality and properties of the materials in the structure and of the factors that influence these properties.

Wood is one of man's oldest and very reliable materials of construction for buildings and bridges. Some highlights of the role of this versatile construction material in the rehabilitation of structures are presented here.

FACTORS AFFECTING SERVICEABILITY OF WOOD STRUCTURES:

To accomplish effectively the task of rehabilitation of existing wood structures, it is imperative that the effects of various factors on the serviceability of such structures be understood. The foremost of these factors is the load a structure must carry, including loads not anticipated in design, such as those of piping, heating units attached to the structural members, ponding, etc. Another important factor is the effect of duration of load. It is well recognised, Forest Products Laboratory (1974), that wood members can absorb short-duration of loads of higher magnitude in relation to design loads of normal duration, Fig. 1. Design codes, such as CSA (1980) classify loads into six categories according to their duration: instantaneous (impact), 1 day (wind, earthquake), 7 days, 2 months (snow), normal (10 years) and continuous (more than 10 years).

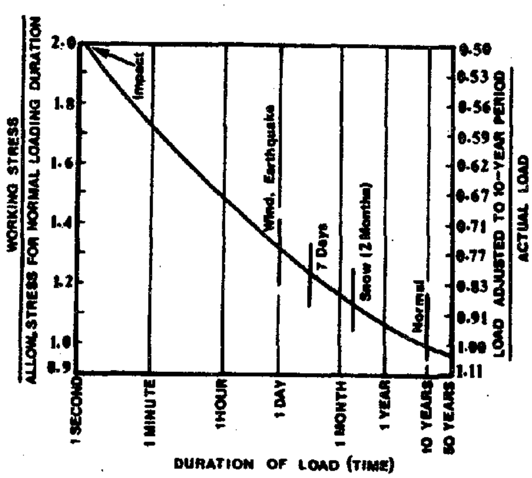

FIGURE 1. Relationship Between Strength and Duration of Load.

Other factors that need to be understood are temperature, moisture, chemicals, weather, decay, destruction by insects and marine organisms and fire. High temperatures, which may occur in structures in the proximity of boilers, have tendency to cause checking, splitting and opening of weak glue joints. Moisture has a significant influence on the strength and some physical properties of wood. Moreover, free water in wood promotes decay, checking, loosening of fastenings and peeling of paint coatings. Some of the stored chemicals have degrading effect around metallic fasteners in wood components. Wood structures exposed to weather are subjected to cyclic effect of wetting and drying, which may cause checking in wood. Wood may be attacked by decay-producing fungi, insects or marine organisms, shortening the useful life of wood structures. Proper construction details and application of preservative treatment, can enhance the durability of the structures.

The loss of load-carrying capacity of wood components under the action of fire is the result of two major causes; namely, the formation of charcoal in the outside portion of the member and the weakening of a thin layer that is immediately beneath the charcoal, Fig. 2. The basic effects of fire on wood member are the

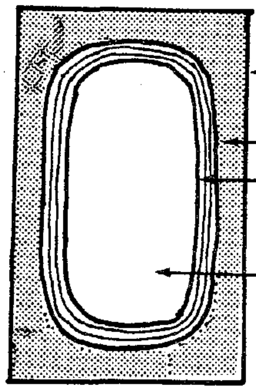

FIGURE 2. Section of Wood After Fire.

reduction in cross section (only cold wood core area is effective) and the weakening of metallic fasteners in the member. Treatment of wood with fire-retardant chemicals or coatings is an effective means of preventing flame spread. Fire-resistance of wood structures can also be imroved through good design and construction details. In heavy timber construction, fire resistance is provided by massive wood and the avoidance of concealed spaces in which fire may originate and spread undetected. As light-frame wood construction, most residential dwellings belong to this group, do not have the fire resistance provided by heavy wood construction, attention should be paid to good construction details to delay the spread of fire and reduce hazards to occupants. Firestops should be provided at exterior walls, at each floor level and at the level where the roof connects with the wall, Forest Products Laboratory (1974). An illustration of the use of firestops in walls is shown in Fig. 3.

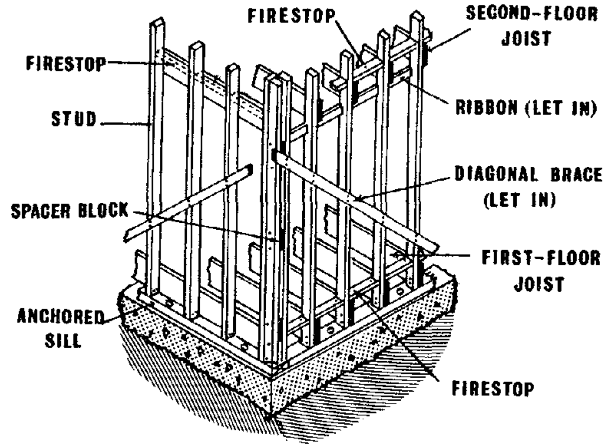

FIGURE 3. An Illustration of Firestops in Walls of Light-Frame Construction.

INSPECTION, INVESTIGATION AND EVALUATION OF WOOD STRUCTURES

As first step in assessing the condition of an existing wood structure, a comprehensive visual examination of all structural components including connections and materials should be undertaken. Inspecting also involves species identification and determination of the extent of deterioration, such as: splits, checks, high moisture content and possible decay at ends of beams and columns; open glue joints between laminations and in end joints; decay in piles; loose fasteners and exposed connectors in trusses; mechanical damage like cutouts and abrasions ; unusual loadings like heavy piping, fans, etc. Many techniques and types of equipments are available for use in the inspection. Some of them are: sonic method, drilling, coring with plug cutter or increment borer, electric moisture meter, X-ray, culturing, microscopy, shell depth indicator, etc.

Structural evaluation of wood construction involves estimating the load-carrying capacity of: structural components that show sign of distress or deterioration; structural members that will be directly affected by the rehabilitation; structural components that are expected to receive increased loading; structural and non-structural components that are affected by earthquake loads. There are three basic methods for evaluating structural components and structural systems: (i) Engineering and scientific judgement based on known past performance; (ii) Load tests; (iii) Engineering analysis. The type of method need to be applied depends on the condition of the structure and the nature of the project.

There are many methods of repair of wood structures, such as: replacement of decayed parts; application of chemicals to arrest decay; structural damage repair by using mechanical fasteners like clamps, bolts, longitudinal rods, etc.; repair of structural damage by adhesives.

Then, there are special techniques applicable to specific problems. A special method of repairing deteriorated piles by the use of reinforced concrete jacket is discussed by Gerke (1969).

AN EXAMPLE OF INVESTIGATION - SEGMENTAL LATTICED TIMBER ROOF TRUSSES

The investigation was carried out on two structures in the Province of Nova Scotia; one was an existing building used for storage and the other, a warehouse, was under construction, Fig. 4. Both buildings had trusses of 60 ft. (18.29 m) span with 8 ft. (2.44 m) centre-rise.

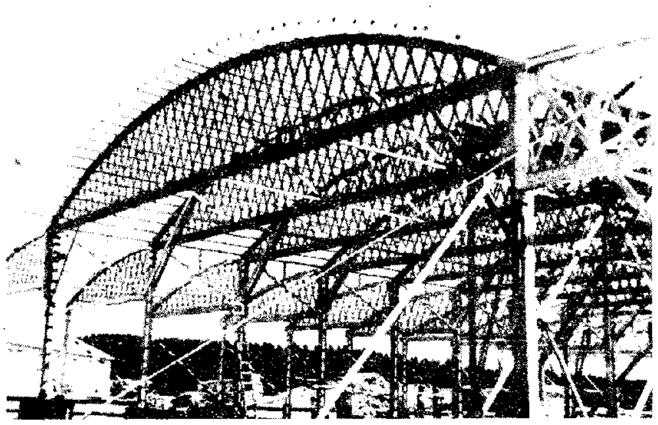

FIGURE 4. Building Under Construction, Using Segmental Latticed Roof Trusses of 60 Ft. (18.29 m) Main Span.

In Fig. 4, side bays are 25 ft. (7.62 m) each. The trusses were spaced 12 ft. (3.66 m) centre to centre. The entire truss was constructed of spruce lumber assembled by nailing principally 3-inch (7.62 cm) common wire nails, but some 1.5 (3.81), 2 (5.08) and 2.5-inch (6.35 cm) nails were also used. Figure 5 gives an overall arrangement of material in the truss members.

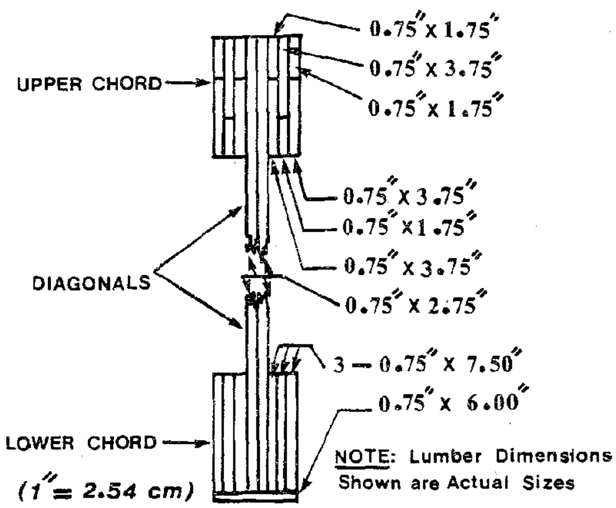

FIGURE 5. Arrangement of Material in Truss Members.

On the basis of the investigation, it was recommended that the existing building not be submitted to full snow load, that is, steps be taken to remove excessive accumulation until upgrading of the structure was done. As for buildings to be erected after this investigation, a number of recommendations were made to strengthen the trusses, including increase in the sizes of members and provisions to upgrade the capacity of the end joints in the trusses. The connections at the supports were rather weak. An experiment was devised with the view to testing the strength of the end connection. A small triangular truss was built so that when load was applied at the centre it had the same forces in the chord members as would occur in the whole truss under a full uniformily distributed load. Figure 6 shows a small truss specimen in test.

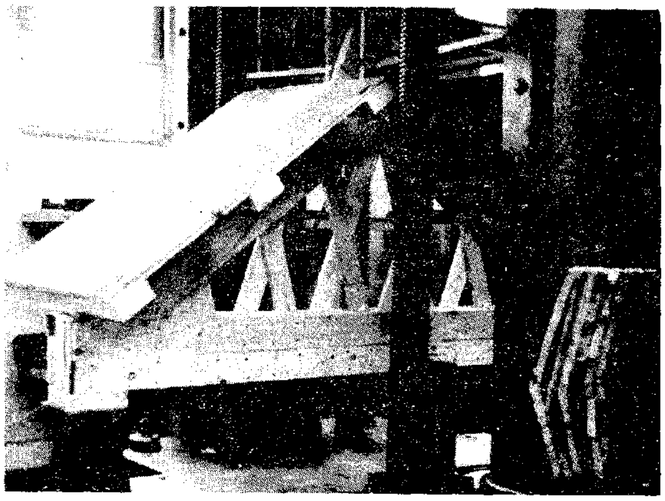

FIGURE 6. Small Test Truss in Testing Machine.

The scope of the above investigation was extended to full scale testing of 60 ft. (18.29 m) trusses. The test roof system consisted of three trusses of 60 ft. (18.29 m) span and 8 ft. (2.44 m) rise, spaced 12 ft. (3.66 m) centre to centre. The locations of gauges for deflection and other measurements during testing are given in Fig. 7. The trusses were loaded by means of

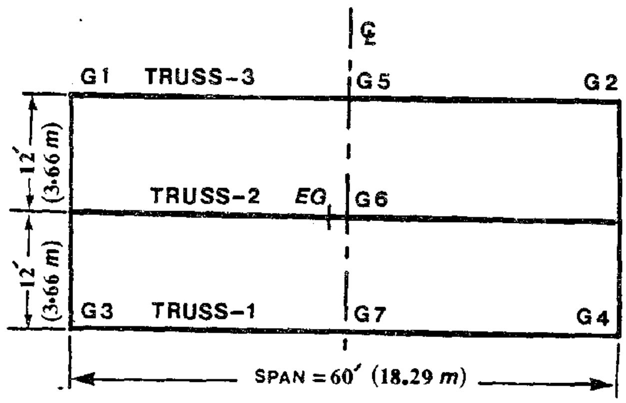

FIGURE 7. Three-Truss Roof System Showing Location of Gauges.

a mobile crane and a skiff, into which plastic bags were placed, each containing approximately 100 lb. (45.4 kg) of sand, Fig. 8. The settlements of the footings were recorded at the extreme corners, locations G1 through G4 in Fig. 7, and deflections of the lower chords were noted at G5, G6 and G7. By using electric strain gauges, strain measurements were made on *four sides of all diagonals in Truss #2* at a section 2 ft.-8 in. (81.3 cm) from the centre line, location EG in Fig. 7.

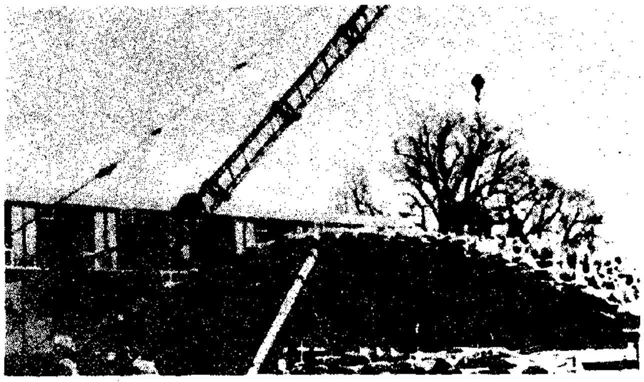

FIGURE 8. Loading on Segmental Latticed Roof Trusses of 60-Ft. (18.29 m) Span.

It was observed in this full-scale test that the segmental latticed roof truss system of the type tested had an ultimate load factor of about 2.76. Details of the tests and test results are given by Mazur and Gilkie (1963). Based on this research and other research and developments projects on nailed laminated timber construction, many development designs for varieties of structures have been prepared as reported by Malhotra and Ritchie (1981).

STRUCTURAL DESIGN CONSIDERATIONS IN REHABILITATION OF BUILDINGS

In recent years, an appreciable interest has been generated in this subject matter. Freas and Tuomi (1980) made a presentation on the ASCE Manual (currently under preparation) on Evaluation,Maintenance and Upgrading of Timber Structures in a recent ASCE meeting. At the same meeting, Peterson (1980) discussed the rehabilitation of a classroom using timber and Silva (1980) presented a paper dealing with wood arch relamination by epoxy grouting. Quaile and Keenan (1979) have recently described some Canadian experiences in the identification of problems in existing timber structures and the design of remedial measures for such structures. Powell (1978) published a paper on reinforcing of structural wood members.

Strengthening of structural systems is done either by strengthening individual structural members or by adding new structural components. The use of combinations of the two approaches is quite common. For example, a wood column can be strengthen with wood. Efficiency curves like the ones shown in Fig. 9 can be used as an aid in the design of such members. This figure gives the variation of ratio of strength of

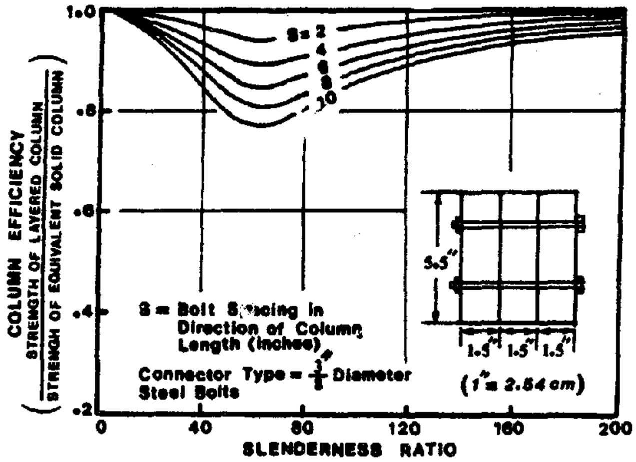

FIGURE 9. Efficiency Versus Slenderness Ratio Curves for Bolted Layered Columns.

built-up column to that of equivalent solid column for different bolt spacings for the column cross section shown on the graph. Equivalent solid column is a column of same overall dimensions as those of the built-up column. Efficiency curves can be developed for various types and sizes of built-up columns with different types and sizes of connectors, based on the research by Malhotra and Van Dyer (1977).

It should be noted that structure may appear to have performed satisfactory over the years, but current code requirements particularly for earthquake loads may require extensive structural modifications. In structural systems for resisting lateral forces created by wind or seismic ground motion, it is pertinent to consider the manner in which the lateral forces are to be transferred to the foundation of the structure. Each structural member must have the strength to resist the applied loads as well be able to transfer those loads to the adjoining elements. Thus, particular attention has to be paid to designing structural connections so as to ensure overall integrity of the structure. An effective structural wood system for resisting lateral forces, is shearwall and diaphragm arrangement. Horizontal roof and floor diaphragms are designed to distribute lateral forces on the structure to the shearwalls. Shearwalls are designed for resistance to racking under the sheathing effect of horizontal loads carried from the top of the walls to the foundations. Fig. 10 illustrates shearwall and diaphragm action in the distribution of lateral forces on a simple box-type building. It should be noted that floors, walls and roofs designed to carry vertical loads only are not necessarily capable of resisting applied lateral loads. If they are to function as structural diaphragm or shearwall, they must be designed adequately to fulfill that role. Care must be taken to adequately reinforce openings in shearwalls and diaphragms with framing members and connections, so that they are at least as strong as the rest of the member. Diaphragms have to be suitably connected to the shearwalls and the entire structure must be fastened to the foundations to resist uplift forces.

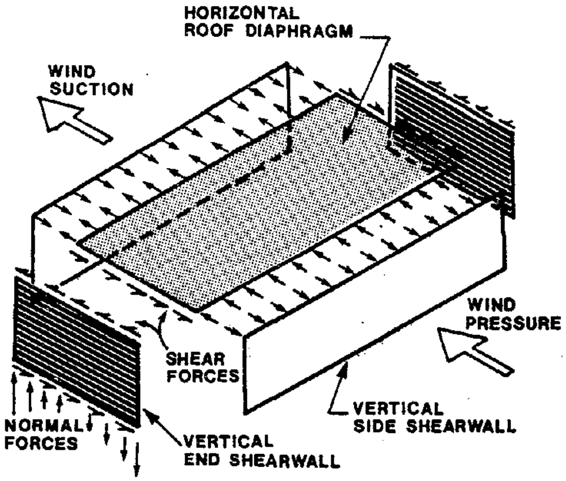

FIGURE 10. Shearwall and Diaphragm Action in a Simple Box-Type Building

In the rehabilitation of structures, one often encounters composite structures made of different materials. Limit states design , a probabilistic design concept currently under development in Canada, provides a unified approach for all civil engineering structural standards and, thus, simplifies the evaluation and design of composite structures. It also offers a means of incorporating safety (reliability) into the invesigation of a structure's limit states. More details on limit states design, particularly as applied to wood construction, are discussed by Malhotra (1981).

SOME EXAMPLES OF REHABILITATION OF BUILDINGS

Historic Properties, Halifax, Nova Scotia

The restoration and rehabilitation of seven historic waterfront buildings in the heart of downtown Halifax, brought a new spirit in the downtown area. The buildings, one of which dates from the early 1800's, make up one of North America's longest surviving group of waterfront structures.

Four of the seven buildings were built in stone in early 188's. All these buildings required a good deal of remedial work to their masonry and timber structures. The other three buidings, built in early 1800's and 1900's, were wooden structures. All three had been severly mutiliated and needed to be completely stripped to their original post and beam timber frame and then totally reclad, reroofed and restored, as closely as possible to their known appearance as of a selected date.

Another 21 buildings in the adjacent streets were included in the rehabilitation project to provide home for an art and design college in downtown setting. The complete project consists of more than 40 retail stores, restaurants, pubs, offices, docking facilities and a college

Butler Square Project, Minneapolis-St. Paul, Minnesota

This project involved rehabilitation of a warehouse type building into eight floors of office and retail space, Horner (1976). It had been awarded an Honour Award by the Minnesota Society of Architects. The remodelling was done by using heavy timber construction - solid timer columns, timber flooring, solid timber joists and beams of two solid timber pieces bolted together. Exterior bearing walls of masonry formed the major lateral stability of the building.

It was determined that the massiveness of the building, 500,000 sq. ft. (46,450 sq. m.), was undesirable for its intended use and consequently the concept of atrium was utilized to provide an openness to the interior space. This required removing a significant portion of each floor and thereby decreasing the structural stability and diaphragm capacity of each floor. The diaphragm capacity of the floor was retored by the use of a new, raised floor and column stability was maitained by use of bolted straps and by retaining or replacing beams as required for lateral stability.

REHABILITATION OF TIMBER BRIDGES

Timber has been used as construction material for bridges for many decades. A recent historical note by Legget (1981) mentions of many railway timber bridges built in Canada in 1880's. Some of these bridges were spectacular in size. Then, there are a number of examples of covered timber bridges over 100 years old still providing service. In recent years, many older bridges in North America have been strengthened and restored for preservation. The reevaluation of the structural soundness of older bridges is essential in the assessment of load limitations to determine suitability as a structure capable of carrying modern traffic loads. A concise guide to the evaluation of the structural adequacy of existing timber bridges is provided by Hurlbut (1977).

An example of use of wood for rehabilitation of a 100-year old trussed iron bridge system in Tioga County, New York, is the construction of a new prefabricated laminated wood panelized bridge system reusing existing piers and abutments, Anonymous (1976). This timber bridge, 290 feet (88.39 m) in span, resulted in substantial saving over other bridge replacement systems. Treated wood used in the bridge provides a low maintenance. Another reason for the low maintenance is that the decking is massive and deflections so small that weakening surfaces would not break up under repeated heavy loads. This panelized timber system has been used in a number of other instances and offers one solution to rehabilitation of many existing bridges.

In recent years, the Ontario Ministry of Transportation and Communications undertook a long-term program, Csagoly and Taylor (1979), to assess the load-carrying capacity of the existing timber bridges and to develop methods for improving capacity as an alternative to replacement. On the basis of extensive field studies and tests, an effective means of post-tensioning of longitudinally laminated timber decks has been developed, Taylor and Csagoly (1978).

The maximum allowable single axle weight in the Province of Ontario has been 22,000 lb. (10,000 kg), but on many mining and logging roads where many of the bridges of the above mentioned type are situated, weights as high as 44,000 lb. (20,000 kg) have been recorded. Under such heavy loads, the conventional nailed laminated bridge decks often fail. In 1976, a small three-span bridge with total span of 55 ft. (16.78 m) and longitudinally timber deck was retrofitted by transverse post-tensioning the deck. The bridge was load-tested before and after the prestressing. After prestressing, the bridge sustained the full specified load without any difficulty. The improvement of behaviour was significant and stresses in the laminates across the deck had been reduced by a factor of 2. Same technique of transverse post-tensioning has successfully been applied to two other bridges, one in southern Ontario and the other, a major bridge, in the Province of British Columbia. A substantial saving has been achieved by the rehabilitation of the three existing bridges with post-tensioning, compared with the cost of replacing them with new bridges.

Huggins and Aplin (1965) carried out an investigation , which included the field inspection of 57 bridges, to determine the extent and possible cause of delamination in existing glued-laminated bridges.

CONCLUSIONS

Wood is a renewable resource, lightweight with its warmth, naturalness and aesthetic appeal, and it is one of the most energy-efficient construction materials. It can be used in varieties of imaginative ways to provide new life to old buildings.

The role of wood in the rehabilitation of structures was briefly discussed in the paper. Various aspects of inspection and evaluation of wood structures were mentioned. Some examples of investigation and rehabilitation of wood buildings and bridges were given. With the use of systematic evaluation techiques, scientific investigations and engineering design, wood structures can be rehabilitated to provide many years of durable service life.

ACKNOWLEDGEMENTS

Acknowledgement is due to the National Research Council and Natural Sciences and Engineering Research Council of Canada for their financial support for some of the projects described in the paper. The author would like to extend his thanks to all individuals, in particular to Mr. R.A.G. Ritchie, associated with various aspects of the studies reported herein for their cooperation and assistance. Appreciation is also extended to Mrs. A. Malhotra for her help in typing and drafting of the figures of the manuscript.

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PART II

REHABILITATION OF BUILDINGS AND RELATED STRUCTURES

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62

EVALUATION AND REHABILITATION OF MULTISTOREY RESIDENTIAL BUILDINGS

G. Robert Fuller **Andrei Gerich**

Structural Engineering Division
Office of Architecture and Engineering Standards
Office of Housing, Federal Housing Administration
U. S. Department of Housing and Urban Development
Washington, D. C. U. S. A.

SUMMARY

During the last ten years there has been expanded interest in renovating existing structures in the United States as an economical alternative to new construction. An increasing number of buildings constructed for various occupancies have been proposed for conversion to residential use under U.S. Department of Housing and Urban Development (HUD) subsidized public housing, and block grant programs.

Several projects in U.S. seismic risk zones, that were originally constructed under lesser restrictive building codes, and designed as hotels or office buildings, were proposed for renovation. However, a methodology to analyze seismic resistance of these buildings was not available to HUD technical personnel or many consulting engineers.

Therefore, in 1976, HUD issued a research contract to develop a "Methodology for Seismic Evaluation of Existing Multistory Residential Buildings." The resultant three-volume manual, contains repair, retrofit and strengthening techniques for masonry, timber, concrete, steel and aluminum structures (Pinkham and Hart, 1978).

This paper is a general presentation of the methodology, applicable strengthening techniques, and evaluation procedure used for HUD projects. The paper will also contain specific descriptions of several multistory buildings in the Los Angeles and San Francisco areas, which were analyzed by private sector consulting engineers and then strengthened to comply with HUD's Minimum Property Standards (HUD, Rev. 1977).

INTRODUCTION

In January 1975 the National Bureau of Standards (NBS) published a study "Natural Hazards Evaluation of Existing Buildings (BSS-61)" (Culver, Lew, Hart, and Pinkham, 1975). This study presented a methodology for evaluation of damage to both structural and nonstructural building components resulting from extreme natural environments such as earthquakes, hurricanes and tornadoes. Three sets of procedures were presented: (1) Qualitative determination of damage level on the basis of data collected in field survey of a building; (2) Determination of damage level as a function of behavior of critical elements based on a structural analysis sis of a building; and (3) Determination of damage level based on a computer analysis of the entire structure.

Based on the second procedure presented by NBS, the U.S. Department of Housing and Urban Development (HUD) in June 1976 awarded a contract to S.B. Barnes and Associates to develop a methodology for seismic evaluation of existing buildings. This resulted in a three volume manual, "A Methodology for Seismic Evaluation of Existing Multistory Residential Buildings" (Pinkham and Hart, 1978) which was published by HUD in November 1978. Contained in the manual are methods of structural analysis, strengthening and repair of existing structures, cost analysis of remedial methods, and examples which illustrate a simplified and a complex (computer) evaluation of stress distribution of different types of multistory buildings.

The HUD Methodology was limited to evaluation of seismic resistance of residential type buildings, but also expanded the BSS-61 procedures by adding strengthening and repair as well as cost analysis to the scope. In addition, a unique computer analysis program was developed.

At present, HUD requires an evaluation of earthquake hazard and seismic resistance of structural components for all buildings located in Seismic Zone 3 (ICBO, 1973), in accordance with HUD Handbook 4940.4, Minimum Design Standards for Rehabilitation for Residential Properties, (HUD, Rev. 1978). See Figure No. 1 for Seismic Zone 3 locations in the United States.

EVALUATION AND STRENGTHENING

There are a number of major cities in the U.S. outside of California which are located in Zone 3, such as Boston, MA; Buffalo, NY; Charleston, SC; Memphis, TN; Salt Lake City, UT; Reno, NV; and Seattle, WA. Therefore, an analysis of seismic resistance of many existing buildings intended for conversion to residential use under HUD programs will be required. Three buildings which have been analyzed and strengthened are described herein.

The manual describes survey and evaluation procedures necessary to determine the seismic hazard of existing multistory residential buildings. The method of evaluation is given in terms of the behavior of the critical structural elements in the building. The determination of this behavior requires an analysis of the structural response of the building to prescribed forces and the determination of the strength of the critical members and connections of the earthquake resisting system of the building. The general methodology is applicable to any complete set of earthquake resisting design standards and to most types of building systems.

The response of an existing building to earthquake motions reflects the performance level inherent in the codes, stand-

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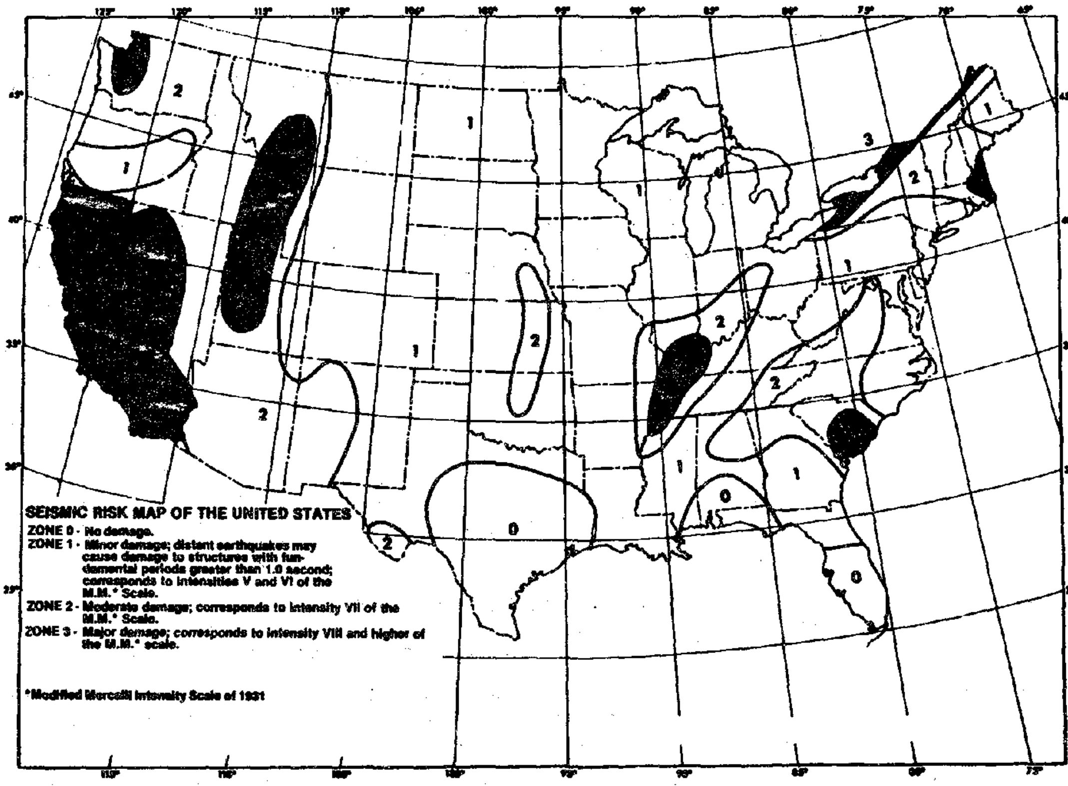

Seismic Risk Map of the Contiguous United States, courtesy Uniform Building Code by International Conference of Building Officials.

ards, and construction practices in existence at the time of the design and construction of the building. Building practices continually improve during the life of a building, reflecting the advancement of the state of knowledge. Thus, the implied margins of safety changes, depending on whether a comparison is made with the code in force at the time of design or with the current code. Deterioration and improper alterations during the service life of the building affect the actual margin of safety provided by the building. The need exists, therefore, to evaluate the potential seismic hazard of each building proposed for rehabilitation. Following such an evaluation, the cost of appropriate strengthening or retrofit procedures has to be calculated so that the feasibility of various schemes to mitigate unacceptable hazards can be determined.

The earthquake effects on a structure that are evaluated using the methodology are those resulting from shaking. Not evaluated are the earthquake effects which produce foundation settlements or soil failures, ground lurching, liquefaction, surface trace of earth faulting, failure of the slope beneath the structure, tsunamis, seiches or inundation resulting from the failure of dams or reservoirs.

The following step-by-step procedure from the Manual was used to evaluate the seismic resistance of the structures:

1. Obtain all basic data and complete Data Collection Forms.
2. Decide, without further analysis, whether or not strengthening is feasible.
3. Extract criteria from UBC 73 and generate necessary basic imput data for analysis.
4. Distribute basic loads from UBC 73 criteria to the structure.
5. Generate appropriate structural model.
6. Determine load effects on critical elements of the structure (axial, flexural, shear and torsional stress on components and connections).
7. Determine resistance capacity of critical components and connections using basic data and criteria.
8. Determine critical stress ratios.
9. Determine whether strengthening is necessary.
10. Identify strengthening procedures and details.
11. Determine costs of strengthening procedures.
12. Decide on whether or not to proceed with strengthening or to provide further evaluation iteration.

COST ANALYSIS OF PROPOSED REMEDIAL MEASURES

Cost analysis of seismic rehabilitation, as contained in the Methodology, are for the purpose of determining:

1. Economic feasibility of the project as a whole, by combining financing costs, other required upgrading costs, costs of temporarily vacating the building, etc;

2. Most economical engineering solution by comparing several alternatives;

3. Level of rehabilitation within given budget restrictions when 100% code compliance is not economically feasible (25%, 50%, 75% code compliance);

4. A budget for future design and construction.

After Data Collection, it may be obvious that major strengthening will be required to bring the structure up to reasonable compliance with the code. The preliminary structural analysis may show that major critical elements are inadequate. From preliminary estimates, the decision can then be made whether or not to proceed with structural analysis and a more detailed cost estimate.

Detailed cost analysis for seismic structural rehabilitation should only include work which falls into one of the following categories:

1. Rehabilitation which contributes directly to seismic structural strengthening,

2. Work required to complete rehabilitation defined in (1), and

3. Work required to return the building to original condition.

These categories that need to be considered in a seismic rehabilitation project are specifically delineated in the Manual.

HUD REHABILITATION PROJECTS

Volume III of the Methodology contains two detailed analyses of actual buildings. Both computer and hand calculations were conducted to determine the resistance capacity of the buildings as they existed.

First, the period of the building was computed; the base shear determined; and loads calculated in conformance with the code. Critical elements were then analyzed and unit stress computed. Next determined was the best possible method of rehabilitation, followed by a preliminary analysis and cost calculation. Finally, critical stress ratios and cost figures for 100% compliance with the code were determined (Cost figures for 75%, 50%, and 25% were also tabulated).

Oakland Hotel; Oakland, California

This was formerly a 400-room hotel which was to be converted to housing for the elderly. It is an eight-story building constructed in 1912 of steel and reinforced concrete.

It was determined by the Structural Engineer that critical elements in this building were the first-floor columns. Critical Stress Ratio (an indicator for evaluting seismic resistance capability of the structure) was 3.6 (28% with respect to UBC '73). The optimum Critical Stress Ratio is 1.0%; therefore, the building would be overstressed by 3.6 times its capacity if subjected to a maximum credible earthquake.

To remedy the situation, reinforced concrete shear walls, 8" to 12" (10.32 - 30.48 cm) thick, were installed on all floors; and a footing supporting three columns was enlarged to prevent overturning.

After remodeling, 315 units of subsidized elderly housing were created. The Cafe, Club Room, Dining Room and Ball Room were retained for use by the residents. Both evaluation and construction have been completed, and the building is fully occupied. Cost of rehabilitation was $14,400,000 ($1,280,000 for the structural work).

Young Women's Christian Association (YWCA): San Franciso

This seven-story reinforced concrete building was constructed in 1931 as a YWCA dormitory. Evaluation of this building using the Methodology revealed that critical elements were first-floor columns and corridor walls. The Critical Stress Ratio was 3.1 (32% with respect to UBC '73).

To increase resistance to earthquakes, the exterior walls were strengthened by pneumatically applied concrete, 4" to 8" (10.16 - 20.32 cm) thick. This strengthening brought the building up to 100% compliance with UBC'73 (Critical Stress Ratio equal to 1.0).

During removal of corridor walls on all floors, large reinforced concrete trusses were uncovered, which indicates that the original designer was aware of the necessity for the building to be capable of resisting lateral forces.

The YWCA building is now fully occupied as housing for the elderly with 98 units. The total remodeling cost was $4,600,000 ($1,200,000 for structural work, including a special soil investigation).

William Taylor Hotel; San Francisco, California

The third building evaluated using the HUD Methodology is the old William Taylor Hotel located at 100 McAllister Street. This 27-story steel frame building with in-fill brick walls was constructed in 1929. For many years the building was used for government offices, but it is now being rehabilitated into a student dormitory for Hastings College of Law.

By making a conservative assumption - disregarding the resistance provided by brick walls but taking into account their weight - the Structural Engineer established the Critical Stress Ratio in beam-column connections as 4.5 (22% with respect to UBC '73). Beam-column connections located at the 14th floor and above were identified as the critical elements. It was, therefore, recommended that framing at the 14th, 21st and 25th floor vertical offsets be reinforced by installing additional steel floors and by providing collector elements.

Evaluation of the building has been completed and demolition work has just started, but the total construction price has not been determined.

CONCLUSIONS

The HUD Methodology is based on a procedure which determines actual damage level as a function of behavior of critical elements. Then recommendations and cost estimates are made to bring the structures up to 25%, 50%, 75% and 100% compliance with the Code. The procedure outlined in the HUD methodology provides the evaluator with the information necessary to arrive at appropriate decisions. The final decision as to the extent of rehabilitation must weigh the risk of loss of life, damage to property and importance of the project against cost of rehabilitation.

Structural Engineers experienced in the design and analysis of structures capable of resisting seismic forces can use their own approach. However, the primary purpose of the HUD methodology is to provide a tool to Structural Engineers not necessarily familiar with aseismic analysis. The fact that some of the most experienced engineers applied its basic concepts underscores its value and usefulness.

The "Methodology for Seismic Evaluation of Existing Multistory Residential Buildings" has proven to be an effective tool in analyzing existing buildings proposed for rehabilitation under HUD programs. Several buildings in the San Francisco and Los Angeles areas have been analyzed and strengthened using procedures outlined therein.

Tabulated cost data related to percent compliance with the 1973 Uniform Building Code has also proven to be invaluable to program administrators and building officials charged with the responsibility of deciding whether to rehabilitate or destroy a building. This data is often used to decide how much strengthening is required and how much is economically feasible. It can also be a major factor in determining acceptable risk.

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A METHODOLOGY FOR EVALUATION OF EXISTING BUILDINGS AGAINST EARTHQUAKES, HURRICANES AND TORNADOES

H. S. Lew

Center for Building Technology
National Engineering Laboratory
National Bureau of Standrds
Washington, D.C. 20234

SUMMARY

A methodology is presented for evaluation of existing buildings to determine the risk to life safety from natural hazard conditions and to estimate the amount of expected damage. Damage to structural building components resulting from the extreme environments encountered in earthquakes, hurricanes, and tornadoes is considered. The methodology has the capability of treating a large class of structural types including braced and unbraced steel frames, concrete frames with and without shear walls, bearing wall structures, and long-span roof structures. Three independent but related sets of procedures for estimating damage for each of the natural hazards are included in the methodology. The first set of procedure provides a means of qualitatively determining the damage level on the basis of data collected in field surveys of the building. The second set utilizes a structural analysis of the building to determine the damage level as a function of the behavior of critical elements. The third set is based on a computer analysis of the entire structure. All three sets of procedures are based on the current state-of-the-art. The procedures are presented in a format which allows up-dating and refining.

Keywords: buildings; damage; disaster; dynamic analysis; earthquakes; hurricanes; natural hazards; structural engineering tornadoes, wind.

INTRODUCTION

Background

Much of the loss of life and property in the United States from natural hazards such as earthquakes, hurricanes, and tornadoes results from the inadequate performance of buildings in response to these extreme natural environments. Past observations have shown, however, that buildings properly designed, detailed, and constructed can withstand these environments. This involves realistic assessment of the forces produced by the environment, proper distribution of these forces to structural and nonstructural building components and providing the necessary resistance to these components in the design process, and continual inspection during construction to insure the design is correctly executed in the field.

Improved building practices incorporating new knowledge relative to natural environments and the performance of buildings will serve to mitigate future losses. Continued updating of building codes and standards, taking into account the latest research findings and experiences gained from past performance of buildings, is one aspect of improved practice. Improved practices, however, apply only to future construction. They do not affect existing buildings. The response of an existing building to an extreme natural environment will reflect the performance level inherent in the codes, standards, and construction practices in existence at the time of design and construction. During the life of the building, building practices continually improve reflecting the advancement of the state of knowledge. Thus, the margin of safety changes from that assumed at the time of design as the state of knowlege and building practices advance. Deterioration during the service life of the building also affects the margin of safety. The need exists, therefore, to continually evaluate buildings with respect to the potential hazard they pose when subjected to extreme natural environmental conditions. Following such an evaluation, appropriate rehabilitation or abatement procedure may be initiated to mitigate unacceptable hazards.

Following recent natural disasters in the U.S., several programs aimed at evaluating the hazard posed by existing buildings have been initiated [1, 2]. Most of these programs are similar in nature, however, each uses a somewhat different method of evaluation. Furthermore, since these methods involve evaluating buildings in accordance with the requirements reflected in current building codes, they do not provide an indication of the risk of explicit levels of building performance in terms of life safety, protection of property, and maintenance of vital functions. They also do not provide an estimate of the amount of building damage to be expected.

This report presents a methodology for survey and evaluation of existing buildings to determine the risk to life safety under natural hazard conditions and estimate the amount of expected damage.

Scope of the Methodology

The natural hazard loading conditions considered in this methodology are those encountered in earthquakes, hurricanes, and tornadoes. While the source mechanism is different for each of these geophysical processes, they all impose dynamic loading. Existing historical seismic and meteorological data are used in the methodology for determining the magnitude of recurrence interval for these hazard loadings. For earthquakes, recoded seismic data were used for determining expected ground motion. For wind loading, annual extreme wind speeds for specific mean recurrence intervals are used.

The types of structures considered include braced and unbraced steel frames, concrete frames, shear wall structures, combination frame and shear wall structures, bearing wall structures, and long-span roof structures. Although no specific limitations are imposed on application of the methodology, it is intended for buildings with substantial occupancy, i.e., fifty or more people. One- and two-story residential buildings, therefore, are not considered.

Three sets of evaluation procedures were included in this methodology, each set representing a different level of analytical sophistication. Hereafter, these three sets are referred to as the Field Evaluation Method, the Approximate Analytical Method, and the Detailed Analytical Method.

In the Field Evaluation Method buildings are evaluated on a qualitative basis in terms of structural characteristics, structural configuration, and the degree of deterioration of the building. Information on these are obtained from a field survey. This method provides a rapid, inexpensive means for identifying clearly hazardous structures or potentially hazardous ones requiring a more detailed analysis to estimate damage.

In the Approximate Analytical Method buildings are evaluated in terms of the behavior of critical structural members. The procedure requires an analysis of the structure to identify critical members and determine the stress level induced in these members by the extreme environments. A set of building plans, specifications, and construction drawings are needed to obtain the necessary data required to perform the analysis.

In the Detailed Analytical Method the damage level is evaluated on the basis of the energy capacity of the structure. This procedure requires the use of a digital computer program for the evaluation. As in the Approximate Analytical Method, the building data needed for input to the computer program would be obtained from a set of building plans, specifications, and construction drawings.

DAMAGE EVALUATION METHODOLOGY

Field Evaluation Method

The Field Evaluation Method can be used where evaluation results do not needed to be refined. This method is particularly applicable if building plans are not available.

Earthquake

For earthquake, a qualitative evaluation of building is made based on a combined effect of structural type, vertical resisting elements, and horizontal resisting elements. Based primarily on the past performance of various types of buildings, relative ratings for these three factors are developed. These are used to determine a basic structural rating which is the basis for determining the building capacity. This is shown schematically in figure 1.

Ratings for various structural types are to account for past performance and the degree of uncertainty that the building would perform in a manner anticipated for the type. These ratings are based on damage experience and judgment. For instance, moment resisting steel frames would be rated good, whereas unreinforced masonry shear walls would be rated poor.

The vertical resisting elements include shear walls, shear cores, vertical bracings, and columns. In assessing the ratings for the vertical resisting systems, "symmetry "quantity," and "present condition" of these individual building elements are considered.

The symmetry describes the eccentricity between the center of mass of the structure and the center of stiffness of the vertical resisting elements. Thus, in a building where the only shear walls are the exterior walls with only one opening in the center of the opposite walls, and the building plan in rectangular, the building would be classified as symmetrical. On the other hand, a two-story rectangular structure with nearly solid side and rear walls, but with front wall almost entirely glazed with only four small piers or columns, would be classified as very unsymmetrical.

The quantity refers to the number of vertical resisting elements. If there are many long shear walls, it would be rated good. In the case of moment-resisting frame structures of steel or concrete, both the strength and the number of columns are important. For closer spacing of columns, usually about 6 m to 10 m bays, a good rating is given.

The present condition describes wall cracks and other damage. Other damage may be damage caused by deterioration from lactic or tannic acid (frequently found in dairy or slaughterhouse facilities) or from severe popping of concrete caused by the use of reactive aggregates. The degree of damage in such cases can only be estimated by visual observation.

For vertical resisting elements, "quantity" and "symmetry are combined as one factor. This factor is then combined with "present condition" to obtain the rating for the vertical resisting elements, see figure 2.

The horizontal resisting elements include diaphragms, premeter beams, and horizontal bracings. The floor or roof systems act as horizontal diaphragms to distribute horizontal forces to the vertical resisting elements, such as shear walls, moment-resisting frames, or braced frames. This diaphragm action is similar to that of a horizontal plate girder spanning between the vertical resisting elements and may be continuous over several supports. In this analogy, the floor itself may be compared to plate girder web, and the marginal beams or walls (chords) compared to plate girder flanges. The floor or roof system, acting as a girder web, is primarily a shear resisting element. The marginal beams or girder flanges (chords) in diaphragm action are primarily subjected to axial loads of tension or compression.

Shears are transferred by the anchorage between floor or roof and the shear walls or frame members. The determination of the adequacy of anchorage of connections involves considerable judgment unless computations are made. For example, where the floor or roof systems are of cast-in-place concrete and are placed integrally with portions of the shear walls or frames, generally a good anchorage with shear transfer capacities can be assumed dependent on the concrete strength, concrete slab thickness and amount and anchorage of reinforcing steel. With metal deck systems, the diaphragm values are dependent on the deck configuration, attachment between units, gage, and attachments to supports.

The capacity of the horizontal resisting system is dependent on either the rigidity of the diaphragm, the anchorage capacity of the diaphragm or horizontal bracings to the vertical resisting system, or the effectiveness of chord members. Thus, the lowest rating of these three factors is considered as the rating for the horizontal resisting system. The rating schemes used to rate the capacity of the horizontal resisting systems are illustrated in figure 2.

The Basic Structural Rating which describes the capacity of a building to resist the earthquake force is obtained by combining the rating for structural type and the rating for either the vertical resisting system or the horizontal resisting system, whichever provides a lower rating. The rating scheme adopted for this study is illustrated in figure 3.

Hurricane

Damage to a building is dependent on two basic considerations. One is the effective wind force, usually in terms of pressure, positive or negative. On the other is the resistive capacity of the structure to lateral forces and also to uplift forces on the roof created by the structure's shape.

The same rating scheme used for structural systems in the case of earthquake can be used for wind in the evaluation of a building's ability to resist lateral forces. Because uplift forces are acting simultaneously with the lateral forces, additional factors such as roof anchorage, anchorage to foundation, and internal pressure should also be considered.

The rating of the building is determined by taking the lowest rating of the "Foundation Anchorage" factor, "Roof Anchorage" factor, or the "Basic Structural Rating" as defined in the case of earthquake since these factors affect the building's capacity to resist wind independently.

Tornado

Damage from tornadoes has been most severe to small, light buildings; although tall, flexible , and heavier buildings have sustained some severe structural damage. The most extensive damage to buildings have been to roofs and exterior claddings, including glass. This includes damage from windblown debris. Because the total effects of tornado on a building is not clearly understood at the present time, only a broad categorical rating of buildings, depending upon their types, is possible. In this study, a poor risk rating is given to small and light buildings. A medium risk rating to small, heavy buildings, and to large, multi-story buildings that could be rated high in wind and earthquake resistance. A good rating can only be given to heavy vault-like buildings known to have been designed for tornadoes.

Approximate Analytical Method

The Approximate Analytical Method provides a simplified analytical procedure for evaluation of the building capability to resist natural hazards by determining stress ratios of critical elements of structural elements. These stress ratios are the ratios of the stresses produced by the loading to limiting stresses of the critical building elements.

For the purpose of this evaluation, elastic analysis of building response will be compared with material design capacities. Design stresses will be those designated by material specifications. In the evaluation, buildings are being analyzed, but not designed. Stresses in structural elements will be checked for the combined effects of lateral and vertical loads. Where lateral loads are included, the combined stresses may exceed code working stresses by one-third, except where not permitted in the specifications, with the provision that the stresses resulting from design vertical loads alone will not exceed code design stresses. This method, in general, does not include the use of a dynamic analysis except in special cases.

Earthquake

Any important earthquake resisting element having the highest unit stress as related to allowable design stresses is a critical element to be considered in the evaluation of the structural system. The term "important element means an element which, if it failed, would seriously reduce the capacity of the structure as a whole to resist lateral forces. Some members would not be critical when deformed beyond their yield level deformations. In other members, yielding may cause an important redistribution of loads. With a multiplicity of well-distributed similar elements, the redistribution of loads would add only a small percentage of stress to adjoining or parallel elements.

In most buildings, the critical elements will be the vertical resisting elements (shear walls or moment resistance frames) and the horizontal resisting elments (diaphragms). This is for earthquake forces acting in the plane of the elements. Earthquake forces normal to a wall are a function of the weight of the wall itself. Where a wall has a long span between floor diaphragms or vertical frame elements, it might be a critical element if its failure would produce collapse of the building as a whole from vertical loads or in-plane lateral forces.

The highest ratio or stress resulting from the required seismic forces (f_e) to the allowable material design stress (f_a) (including 1/3 increase where permitted but deducting capacity required by gravity loads) on any critical element is termed the critical stress ratio f_e/f_a. The critical stress ratio is the indicator for evaluating the seismic resisting capability of the structure. Load factors and ultimate capacities are used for concrete design and for plastic design of structural steel.

Hurricane

Both internal and external pressure must be considered on portions of buildings such as roofs and walls. Corners of walls are exposed to high negative pressures. These corners should be checked.

In determining the response of the building as a whole, the type of diaphragm system must be considered. A stiff diaphragm will distribute horizontal forces to vertical resisting elements in proportion to their relative rigidities. A flexible diaphragm will distribute forces to vertical elements more nearly proportioned to the tributary-exposed wind surfaces.

The usual critical elements for wind resistance are, as in earthquake resistance, the vertical resisting elements (shear walls, braced bays, or moment resistance frames) and the horizontal resisting elements (diaphragms). There is a major difference, however, in that wind forces are applied to exposed surfaces while earthquake forces originate at centers of mass and are proportional to mass. Thus, a lightweight exterior wall might have a relatively small earthquake force normal to the wall but would be exposed to wind forces which are independent of the weight of the wall.

The calculations for the adequacy of the building to resist wind forces can be made using any of the standard analytical procedures. The lateral loading to each story level is determined with positive pressures on the windward side and negative pressures on the leeward side. The path by which these forces are transmitted to the vertical resisting elements is determined and the adequacy of diaphragm or horizontal bracing system to transmit these forces should be evaluated. Lateral forces are applied to the vertical resisting elements at each level and the stresses should be analyzed. The overturning stresses should be checked, including uplift on foundations.

The highest ratio of stress resulting from the wind forces applicable to the site (f_w) to the allowable material design stress (f_a) (including 1/3 increase where permitted but deducting capacity required by gravity loads) on any critical element is termed the critical stress ratio f_w/f_a.

Tornado

For the purpose of evaluation, it will be assumed that a free field wind velocity of 200 miles per hour and a pressure drop of 1.2 psi. Using the formula $P = .0025\ V^2$ to convert velocity to pressure gives $P = 100$ psf. The 1.2 psi pressure drop is, converting units, equal to a suction or uplift of 172 psf. If it is assumed that tornadoes are similar to other high winds, such as hurricanes, with respect to the relationship of pressures on windward and leeward sides, one finds these coefficients of the velocity pressures to be 0.8 and either 0.5 or 0.6, respectively; depending on the height-width ratio of the building. Thus, the total lateral force on a building subjected to tornadoes for a moderate degree of protection will be 100 x (0.8 + 0.5 or 0.6) = 130 or 140 psf.

There are probably very few buildings that will be undamaged if in the direct path of a strong tornado. The least likely to be severely damaged will be heavy reinforced concrete or reinforced masonry vault-like structures one or two stories in height, with relatively heavy and solid walls. Taller buildings designed to resist hurricanes may have limited damage, but probably will not collapse unless they are of unusual configuration or have large roof overhangs or open sides. Light buildings of wood frame or steel frame and metal sidings have been known to have been torn from their foundations and blown considerable distances. It is possible, however, it provide such light buildings with some resistance to winds by proper anchorage to foundations to the ground to resist uplift.

If a building has been evaluated for wind and given a poor rating, it will be considered to be inadequate to resist tornadoes. The ability of roof systems, designed only for gravity loads, to resist reversals from uplift need to be evaluated. The wall capacities must be checked for direct horizontal positive and negative force. The floor and roof systems must be checked for adequacy as diaphragms to transmit lateral forces to the vertical resisting elements. Anchorage of walls to footings should be checked for capacity to resist sliding combined with vertical uplift and overturning. The footings themselves should be checked for sliding resistance, uplift, and overturning forces. In checking these the purpose is to determine the ratio of stresses resulting from the imposed tornado loads to the capacity of the structure.

In each of the various resisting systems, such as roof, floors, shear walls, or moment-resisting frames, there may be one or more critical elements which will fail before the other elements. Care must be taken not to derate a building because of one non-important element. Where the failure of such an element would not cause failure of a system, but would only cause minor redistribution of loads, such a member would not be a critical element. The procedure for determining the critical stress ratio is the same that was used in the case for hurricane.

Detailed Analytical Method

This method is based on a modular computer program with each module dealing with a particular aspect of the damageability prediction problem which includes:

1. Environmental Loads
2. Structural Characterization
3. Response Computation
4. Estimation of Potential Damage

For seismic loads, historical and recorded data may be used along with the program describing the seismic and wind activity for the geographical location. Historically based tornado and hurricane activity should be included. With these data, the program will computer, in a probabilistic sense, the specific environment of any given building site in the country. Alternatively, the user may choose to input any of these loads directly.

Structural models of varying complexity can be generated depending on the availability of structural data and the level of effort selected for a particular task. Damage predictions are made on the basis of the building's response to the appropriate loading conditions. Damageability data characterizing the capacity of the building to resist failure must be input by the user. Algorithms for computing damage are based on the assumptions that percent damage varies continuously with key response parameters. These key parameters have been selected and are incorporated in the program. The forms of damage distribution curves as functions of response are also built into the program. The user may choose values of the parameters of these functions in accordance with prepared guidelines and exercise his judgment according to the application at hand. A schematic presentation of the computer program is given in figure 4.

Natural Hazard Loading

Earthquake Loads

Site loads for an earthquake are defined by a ground site response spectrum. This response spectrum reflects an amplification of the hardrock spectrum for the site, which is frequency dependent. The hardrock spectrum is generated in either of two ways. If the risk option is selected, the user must input seismicity data from the Seismic Map provided with the computer program. By further specifying either a return period or building life and probability of non-occurrence, a risk earthquake defined by its Richter magnitude is computed. If the risk option is by-passed, then the user must input a Richter magnitude and a hypocentral distance. In either case, the computer program computes the maximum hardrock acceleration, velocity, and displacement at the site.

Hurricane Loads

If a wind velocity is generated for the specified site, then a statistical regression analysis is used to compute a mean wind velocity given either a return period or building life and probability of non-occurrence. A standard deviation is also computed and may be used to establish some degree of confidence in the wind velocity selected for analysis. If the user desires, he may alternately choose to input a wind velocity for analysis.

As in the case of earthquake, local site conditions modify the wind velocity which is determined on the basis of historical data. For example, free-field wind velocities in large cities are attenuated by the presence of buildings, whereas, on an open prairie, the attenuation is comparatively small as one approaches the ground. Thus, in figuring the wind load on a building, surface conditions are accounted for by specifying a site condition parameter.

Tornado Loads

The proabability of being hit by a tornado can be computed from historical data. However, the user must specify a tornado wind velocity. In this case, the static response of the building will be computed and damage assessments made.

Structural Characterization

Three general levels are offered for structural dynamic modeling:

1. A Detailed Model

2. A Story-Stiffness Model

3. An Empirical Model

It is noted that all of the modeling options lead to the same basic dynamic characterization: building natural frequencies and modal displacement:

The three modeling options are illustrated in figures 5, 6, and 7. If the user selects "Detailed Model" option to generate a detailed stiffness matrix for the structure, then the stiffness matrix is constructed frame by frame, story by story, from top to bottom. Each frame is parallel to the direction of motion in a vertical plane. Stiffness and mass contributions from each frame are superposed in formulating the two-dimensional model of a building. If the user desires "Story-Stiffness Model" option, he must input a story stiffness and story height data for each building story. A total story-stiffness is the sum of the stiffness produced by all columns, partitions, walls, and other lateral force resisting components at that story. If an empirical model is to be generated, then the user must input an estimate of the building's fundamental period and story heights. A straight-line mode is assumed to compute deflections.

Response Computation

Response computations are made for earthquake loads, for tornado and hurricane loads, and for uplift due to wind. Ponding loads are also computed in the long-span roof subroutine.

The response of a building to earthquake ground motion is evaluated by determining the peak modal response in each of the modes (a maximum of six is considered), and combining their contributions to the total response. A damping, a ductility factor, a modal combination scheme, and a value for each story's drift-to-yield are used to compute an effective ductility for the building. An iterative procedure must be used to establish consistent values of damping and response as damping is defined in a function of ductility. While this operation seems to be stable and typically converges in a few cycles (three or four), and upper limit (e.g., six) on the number of interactions is put in and used to transfer control out of the iterative loop in case the computations do not converge to within five percent accuracy. This may only occur when elastoplastic response is considered.

Inter-story drift, pseudo-velocity, and absolute acceleration are computed for each story. In addition, story forces and shears are computed.

The response of a building to wind is treated as a static problem. The force acting along the building is computed by multiplying the pressure at each story by the tributary story area. For the detailed and story-stiffness models, the inverted stiffness matrix is used to compute deflection of the building. If the empirical modeling option is chosen, the building response is calculated considering the building to be a uniform cantilever beam using a simplified analysis.

Evaluation of Damage

Potential damage to a building which may result from exposure to the environmental loads computed for the building site is evaluated in a damage subroutine. Damage is expressed in percent of total damage on a story-by-story basis. Damage is computed independently for earthquake, hurricane, and tornado. It is segregated into three categories: structural, non-structural, and glass. In the case of structural damage, the damage is further subdivided into damage to frame, walls and diaphragms.

The key response parameters used to predict damage in each case are outlined below.

Earthquake

a. Structural: Interstory Drift
b. Non-structural: Floor Velocity or Acceleration
c. Glass: Interstory Drift

Wind, Tornado, or Hurricane

a. Structural: Interstory Drift
b. Partitions: Interstory Drift
c. Glass: Direct Pressure

SUMMARY

This report presents a methodology for evaluating the potential damage of buildings due to earthquake and extreme wind including tornado and hurricane. Three independent, but related, sets of procedures are developed. These ranged from a qualitative procedure based on field survey data to a detailed analytical procedure involving a digital computer program. Historical seismic and meteorological data are used as the basis for establishing environmental loads. Damage estimates are based on empirical correlations between structural response and observed damage coupled with engineering judgment.

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[2] "Development of Methodologies for Evaluating the Earthquake Safety of Existing Buildings," NTIS Report No. PB 267354, National Technical Information Service, Springfield, VA.

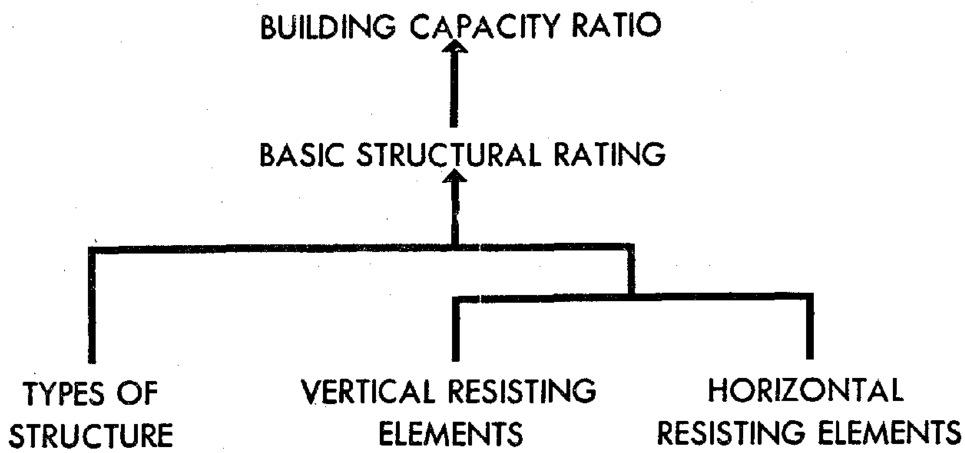


Figure 1. Scheme for Determining Building Capacity

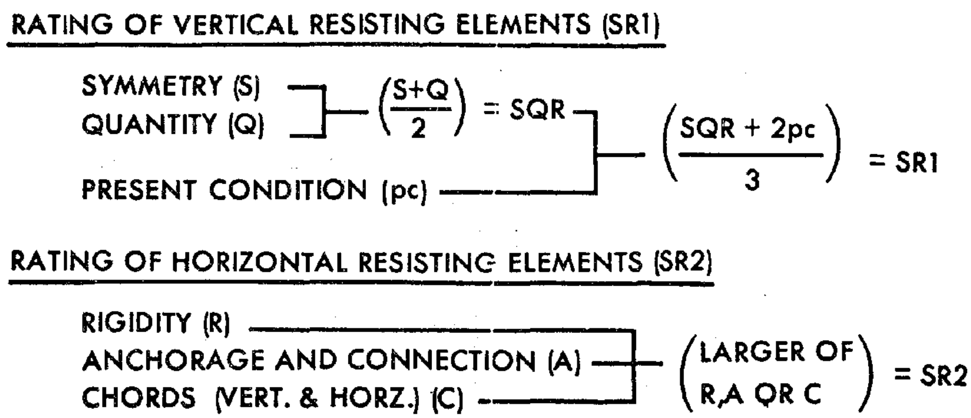


Figure 2. Rating Schemes for Vertical and Horizontal Resisting Elements.

BASIC STRUCTURAL RATING (BSR)

$$BSR = \frac{GR + 2\ (\text{LARGER OF SR1 OR SR2})}{3}$$

RATING OF TYPES OF STRUCTURE (GR)

Figure 3. Scheme for Determining Basic Structural Rating

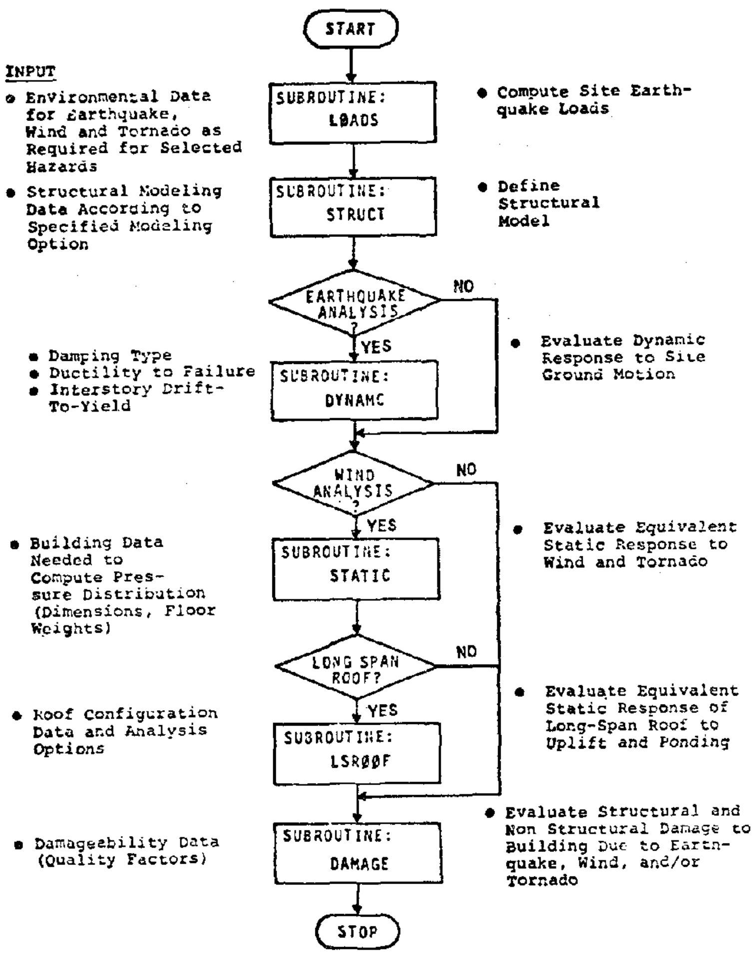
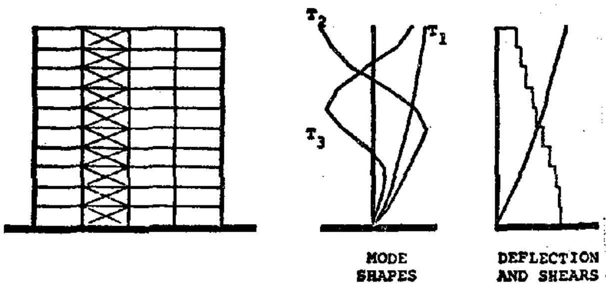
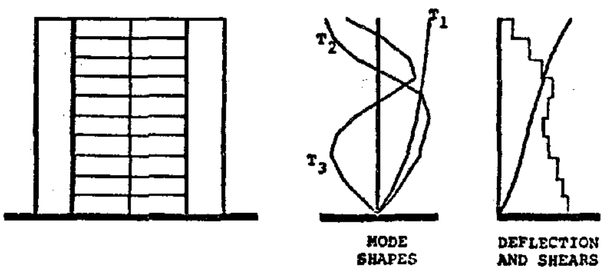


Figure 4. Schematic of Computer Program



UNBRACED OR BRACED FRAME BUILDING



SHEAR WALL OR FRAME AND SHEAR WALL BUILDING

Figure 5. Detailed Frame Model Options

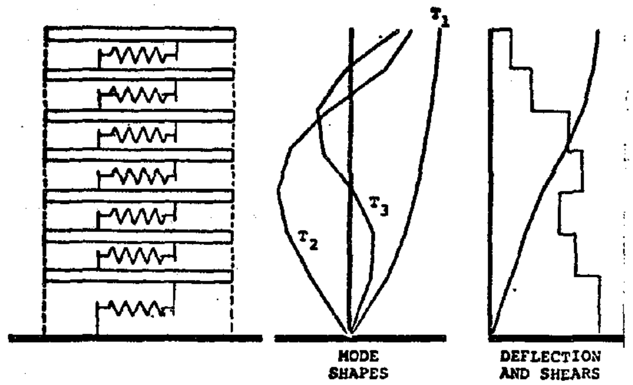


Figure 6. Story Stiffness Model

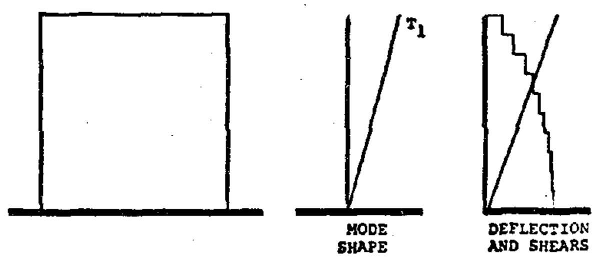


Figure 7. Empirical Model

ASSESSING THE SEISMIC VULNERABILITY OF DIFFERENT TYPES OF EXISTING HOUSING STRUCTURES

Daniel L. Schodek
Department of Architecture
Graduat School of Design
Harvard University
Cambridge, MA. U. S. A.

SUMMARY

This paper describes a method for assessing the overall seismic vulnerability of an urban area in an earthquake-prone region and for targeting groups of structures that are particularly susceptable to extensive damage. Results can be used for disaster-planning purposes since damage levels, injuries and deaths, and loss of habitation are known to be interrelated. The method also provides a rational basis for targeting policy response measures appropriate for implementation during building rehabilitation efforts which are designed to mitigate hazards through retrofit seismic strengthening programs and other measures. Preventive measures need not be applied to entire urban areas, but only to targeted groups of buildings; thus making such programs more cost effective and viable.

INTRODUCTION

A considerable amount of the world's housing stock exists in earthquake prone areas. In most regions of this kind, the existing housing stock was not specifically designed and constructed to resist the forces introduced by earthquakes. The extent of life and habitation loss in recent years is known to be severe (Gauchat and Schodek, 1980). Not only is this issue critical from a disaster-planning perspective, it is also crucial to the architects, engineers, and planners concerned with the use and possible rehabilitation of existing buildings.

Of specific interest herein are the needs of those professionals concerned with developing public policy measures, applicable to aggregate groups of buildings, which are designed to mitigate life and property losses through programs designed to introduce seismic strengthening into existing buildings at the time of rehabilitation. Typical policy devices range from building code measures through point-of-sale requirements for securing financing. Difficulties in implementing such measures, e.g., economic arguments, however, invariably surface.

It is argued in this paper that such programs would have a better chance of success if they were targeted to be applicable only to those specific groups of buildings known to be especially hazardous. Most programs attempted to date have an "across the board" application to all buildings in the area of concern which is both unnecessary and serves to generate widespread resistance to implementation of the programs.

In order to target programs, however, it is necessary to have a technique for identifying groups of buildings particularly susceptable to seismic damage and which consequently pose special threats to life and property. Useful information for formulating programs for a particular locality include a knowledge of the probable extent and geographic distribution of the damage that could be expected in a hazard prone area in terms of both the actual damage levels possible, e.g., light, moderate, heavy severe, or collapse, and the probable number and location of structures experiencing each of these different damage levels, Of comparable importance is data on the relation between the total number of structures in an urban area exposed to an earthquake and the number which would actually experience damage. (Due to local variations in seismic intensities and in soil or geological conditions in an area, as well as different inherent structural resistivities of different housing types, it can be expected that only a portion of the total population of housing structures in a stricken area would experience significant damage.)

The remainder of the paper descnibes one method for achieving these ends and speculates on the possible planning policy implications of having this type of data available.

OBJECTIVES OF STUDY

The research described in this section focused on existing housing structures only, although the methodology developed is suitable to other building types as well. The study is briefly described below and documented in detail elsewhere (Gauchat and Schodek, 1981).

The specific goals of the research were as follows:

A. to provide a method for identifying the range of different housing types (from single family buildings through various types of multifamily structures) that exist in any seismically hazardous urban area, for quantifying the numbers of the different types identified, and for mapping their relative geographic distribution;

B. to provide a qualitative assessment of the relative seismic vulnerability of the housing types found to be prevalent in hazardous areas, but whose seismic performances might be largely unknown (probable damage states as a function of varying seismic intensity levels were identified);

C. to provide a method for mapping potential housing damage distribution patterns in a hazard-prone urban area on the basis of mapped distributions of different housing types, a knowledge of their relative seismic intensity levels, geological conditions or soil types.

DESCRIPTION OF OVERALL INVENTORY AND DAMAGE ASSESSMENT METHOD

The method developed consists of several major phases which can be applied to any urban area. Boston, Massachusetts, in the U.S. was selected as an example study area to illustrate the process described. The steps are as follows:

Development of Data Bases

The primary method for determining the nature of the housing stock present in an urban area found most suitable was the analysis of readily available aerial photographs. Census, tax assessment, and other data bases, however, can be used as well to supplement data obtained from the aerial analysis.

Housing Type Identification

Although communities normally contain many housing forms, a closer examination of the housing in a given community typically reveals the existence of recurring generic types (particularly in the medium to low density range). Some prevalent types can be isolated. Common in the United States are single family detached and semi-detached houses, various kinds of row housing, and high- and low-rise structures with single- or double-loaded corridors or point access circulation networks. A range of concommitant variants have evolved for each generic housing type due to the differences in contextual constraints and local building methods. Different types than those mentioned are, ofcourse, found in other countries.

Standard photogrammetric techniques were used to analyze aerial photographs with the intent of identifying the types and distributions of types present in the study area. This analysis was predicated on the development of a special housing typology which defines and classifies prevalent housing types actually found in the study area. Housing typologies used to date are largely based on the type of dwelling unit access network present. Dimensional and feature analysis techniques were used to identify different aerial images.

Mapped Distributions of Housing Types

Data gained from the housing type analysis was next mapped. This step required the gathering, manipulation, and analysis of large amounts of data defining the types and distributions of housing found present in the study area. The ODYSSEY system, a computer based cartographic data system, which was developed at the Laboratory for Computer Graphics and Spatial Analysis at Harvard University was found especially useful for these purposes. The ODYSSEY system is functionally defined, embracing one particular type of data transformation. All modules communicate with one another using external media, typically sequential data files representing either polygons, chains, segments, data points, or data values. The system facilitates the manipulation of spatial data. Data overlays and selections are easily accomplished (Laboratory for Computer Graphics, Harvard University, 1981).

For purposes of this study, the following general housing types were mapped: one and two family detached units, "three deckers" (a form of three family, one unit/floor, walk-up building unique to the Northeastern part of the U.S.), row houses, single and double-loaded corridor buildings (multiple dwelling units/floor, walk-up and elevator served), point access buildings (multiple units/floor, walk-up and elevator served).

Figure 1 illustrates the general type of map generated. Only a small portion of the overall study area is shown. Note that the areas delineated are ground coverage areas for different housing types (and not individual buildings). The map shows only housing areas. Other building types exist in the area as well.

Analysis of Housing Type Construction

In order to later assess their relative seismic vulnerabilities, a study of the construction of the different types of housing found to exist in the study area was made. In general, the one and two family buildings were made of wood frame construction, and the three-deckers a combination of wood stud infill with post and beam mill framing. Row houses were made of load-bearing masonry walls with wood joist floors. Low rise corridor and point access walk-up buildings were typically load-bearing masonry construction while their elevator-served counterparts were typically steel or reinforced concrete frames enclosed with non-load bearing masonry walls. Many exceptions to these trends, ofcourse, exist.

Detailed construction studies were made of each type and results carefully documented. Details which were felt to be important from a seismic performance viewpoint were especially included.

Seismic Vulnerability Analysis

Based on the previously obtained construction documentation, assessments were next made of the probable seismic performance of the housing types

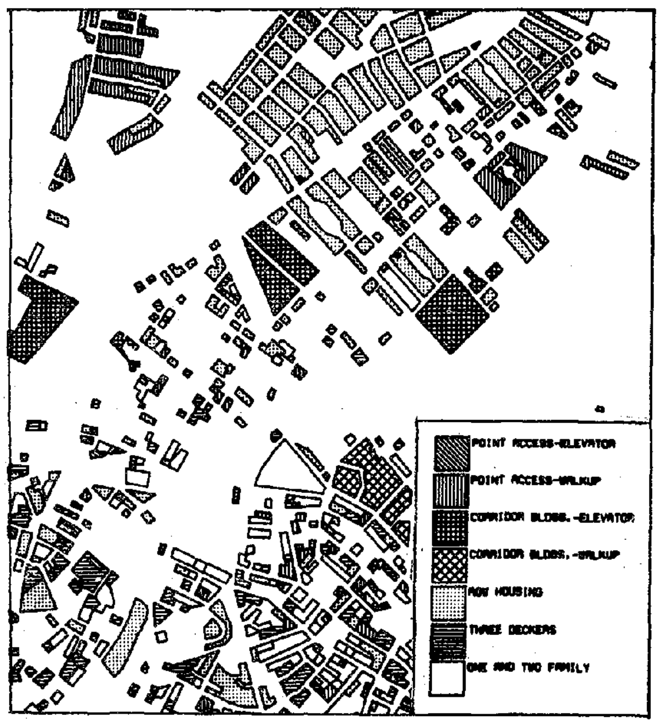


Figure 1. Computer-Generated Map Showing the Distribution of Housing Types in Boston, Mass., USA (Sample Portion).

identified as most prevalent in the study area.

This is a difficult step that can be accomplished in one of several ways. The most traditional technique is through standard engineering analysis procedures. In their simple andmost useable form, however, these techniques are not particularly appropriate for use with the inelastic structures normally encountered in indigenous housing. Advanced techniques are time-consuming and require that many assumptions be made about how certain elements would respond during earthquake conditions. Results are often tentative at best. This approach also does not shed any light on the next required step of extrapolating results from one analyzed unit to a family of units. Empirical data of this type are also very sparse.

In view of the difficulties involved, a simpler approach was adopted which relied upon the collective professional opinion of experienced engineers and architects familiar with both the housing of concern and common forms of earthquake damage.

For the example study area, a questionaire was prepared and sent to experienced engineers. The questionnaire was divided into three primary parts. The first part dealt with establishing definitions and background information, the second with the structural behavior and subsystem damage states for each of the different housing types for a specified seismic intensity, and the third with comparative evaluations of overall damage states for all of the types for a range of seismic intensities.

Developing appropriate damage state descriptions proved difficult since it was desired to have a single set of descriptions which would allow a user to respond both to questions about damage in specific building components, e.g., floor systems, and complete building assemblies. The level of detail and appropriateness to residential construction was also crucial. Most damage descriptions reviewed in the literature were largely developed for comparisons among a range of building types, and contained relatively brief descriptions that did not really address many of the possible damage

Figure 2. Definition of Housing Damage States(Results of Questionnaire).

DAMAGE STATE	DAMAGE STATE DESCRIPTION(ABBRIEVIATED)
NONE TO MINOR (N)	NO DAMAGE OR MINOR NON-STRUCTURAL DAMAGE: Localized cracking is present in some wall finishes, e.g., sheetrock, and in some joints of masonry walls. Some doors & windows misfit or jam because of frame warping.
LIGHT (L)	LIGHT STRUCTURAL AND NON-STRUCTURAL DAMAGE: Occasional minor pulling apart of wall, floor & roof planes. Widespread cracking is present in some wall finishes,e.g., sheetrock. Minor pulling apart of porches from the primary structure. Movement in water and waste lines resulting in some leakage. Sliding or shifting of some floor-mounted mechanical equipment. Minor pulling apart of plumbing fixtures from bases or wall attachments.
MODERATE (M)	MODERATE STRUCTURAL AND WIDESPREAD NONE-STRUCTURAL DAMAGE: Floor or roof planes sag or deflect extensively in several areas. Walls show some evidence of buckling. Minor shifting out-of-plumb of walls is evident. Floor or roof planes show some evidence of buckling. Widespread cracking is present in wall finishes, joints in masonry, and masonry units. Chimneys are shifted out of alignment to limited extent. Widespread door and window misfits and jams are present. Extensive pulling apart of plumbing fixtures from bases or attachments.
HEAVY (H)	HEAVY STRUCTURAL DAMAGE AND NON-STRUCTURAL DAMAGE: Widespread separation of wall,floor & roof planes to the extent that bracing is necessary to prevent post-earthquake hazards to occupants.Chimneys & parapets tilt in precarious and potentially unstable ways. Buckling of support legs of floor-mounted equipment. Separation of electrical wiring resulting in an interruption of electrical service. Separation or cracking of gas lines resulting in a major fire hazard.
SEVERE TO COLLAPSE (C)	SEVERE DAMAGE OR COLLAPSE: Complete or partial collapse of wall, floor and roof elements. The entire framing system is racked or is shifted or twisted off its foundation to the extent that blocking or bracing is necessary to prevent post-earthquake hazards to building occupants. Parapets and chimneys are collapsed. Exit doors are blocked or jammed.

types important in residential construction, e.g., twisting of frames on foundations, and were consequently found too general for the specific needs at hand.

The approach finally adopted was as follows. An extensive listing was developed of possible damage types for each of the major subsystems in residential construction. Each damage type was given an identification number. Based on a review of comparable efforts recorded in the literature and on the basis of professional judgment, each of these descriptions was then placed in one of five damage states. For convenience, the general groupings are briefly described as follows: "None to Minor", "Light", "Moderate", "Heavy", "Severe to Collapse", (N, L, M, H, C,). An abbreviated verbal description was then prepared. The actual definition of each of these damage states, however, was the full compliment of damage descriptions assigned to the particular damage state.

Review of the resultant damage state definitions then became the first question in the actual questionnaire. Respondents were asked to review the actual damage descriptions and their respective assignment to one of the five damage state levels. The results of this process are shown in an abbreviated form in Figure 2.

The participants in the survey were then asked to assess the probable seismic behavior of the different housing types studied with respect to the damage state definitions previously established. This was done for a range of seismic intensities. A choice was made to use the Modified Mercalli Intensity (MMI) scale despite its known shortcomings and definition as a damage descriptor (thereby introducing a form of circular reasoning) because of the fact that most historical data for the housing types studied was documented in this form. Relations to other measures were subsequently established. Results of this survey were incorporated directly into the maps described next.

Damage Distribution Maps

The data derived from the vulnerability study discussed above was then combined with available maps (not shown herein) defining the expected distribution of seismic intensities in the study area into "damage distribution maps". These are maps which show the expected damage states in housing which are likely to occur in different parts of a city after a seismic event. Hence they are forms of scenarios, useful primarily for policy planning purposes. This exploratory effort was generally successful in terms of developing a method for generating maps of this type. It demonstrated, however, that more extensive data than was actually available is needed for results to be convincing. The maps were made using the ODYSSEY system discussed earlier.

One example is shown in Figure 3. This map shows the location of housing likely to experience the different damage states noted in the event of earthquakes of MMI = VIII. The specific areas delineated in Figure 3, it should be noted, show only average or mean damage states. Within any zone, it can be expected that there will actually be damage spread occuring among the family of building covered in which different percentages of structures within the zone will experience greater or lesser damage states. But the expected average damage state is that indicated. Zones indicated, by the way, do not represent only one building class but may represent multiple classes which have similar average damage states. (Other studies are underway which are intended to define the way damage will actually be spread among the groups of buildings that are illustrated as having a given average damage state). The maps cannot be used to identify damage to any particular building.

Clearly there is a wide variation in damage expected. Only light to moderate damage is expected in the southern part of the study area. Considerable higher damage levels are expected in the inner city areas.

While not reported herein, it is possible to use the ODYSSEY system to generate estimates of the actual numbers of buildings that would experience any of the damage states illustrated. Exposed populations can be estimated as well.

The damage distribution map shown was generated by using the data manipulation and overlay capabilities of the ODYSSEY system. A values file was first created for the primary housing cartographic data base. The values file related housing type to intensity level to damage state.

Differences in intensity levels (hence damage states) expected because of variations in seismic intensities in the study area are handled by first overlaying an expected intensity map with the housing map. A new file is created in which polygons are identified by both housing type and intensity zone. Th us the locations of all housing types in d fferent seismic zones are computed. Relating this new polygon file to the values file defining expected damage states allows damage distribution maps to be generated.

APPLICATIONS

Disaster Planning

Maps of the type shown can be used for life-line disaster-planning purposes since areas expected to be hardest hit, i.e., areas experiencing the greatest damage and hence probable life losses, injuries, and losses of habitation can be predicted.

In this context it may be useful to simplify the complex damage distribution maps into more general planning zones which more or less define regions of comparable damage levels. For many planning purposes, however, it may be desirable not to split natural zones and ascertain expected mean damage extents within each of the zones. Different zones would thus have different expected mean damage levels and different populations and numbers of dwellings, but natural groupings would be maintained. Priorities for disaster relief or assistance could then be assigned based on this type of data. Figure 4 shows a map of this general type. Zones A and B are most critical due to the combination of high concentrations of dwelling units and population with high levels of expected damage. F and H are least critical (F because of the small exposed population and number of dwelling units and H because of the low damage expected).

Prevention Measures

The damage distribution maps can also be used as a way of identifying what regions of a city are most vulnerable to damage and consequently

PROBABLE DAMAGE STATES
Modified Mercalli Intensity = VIII

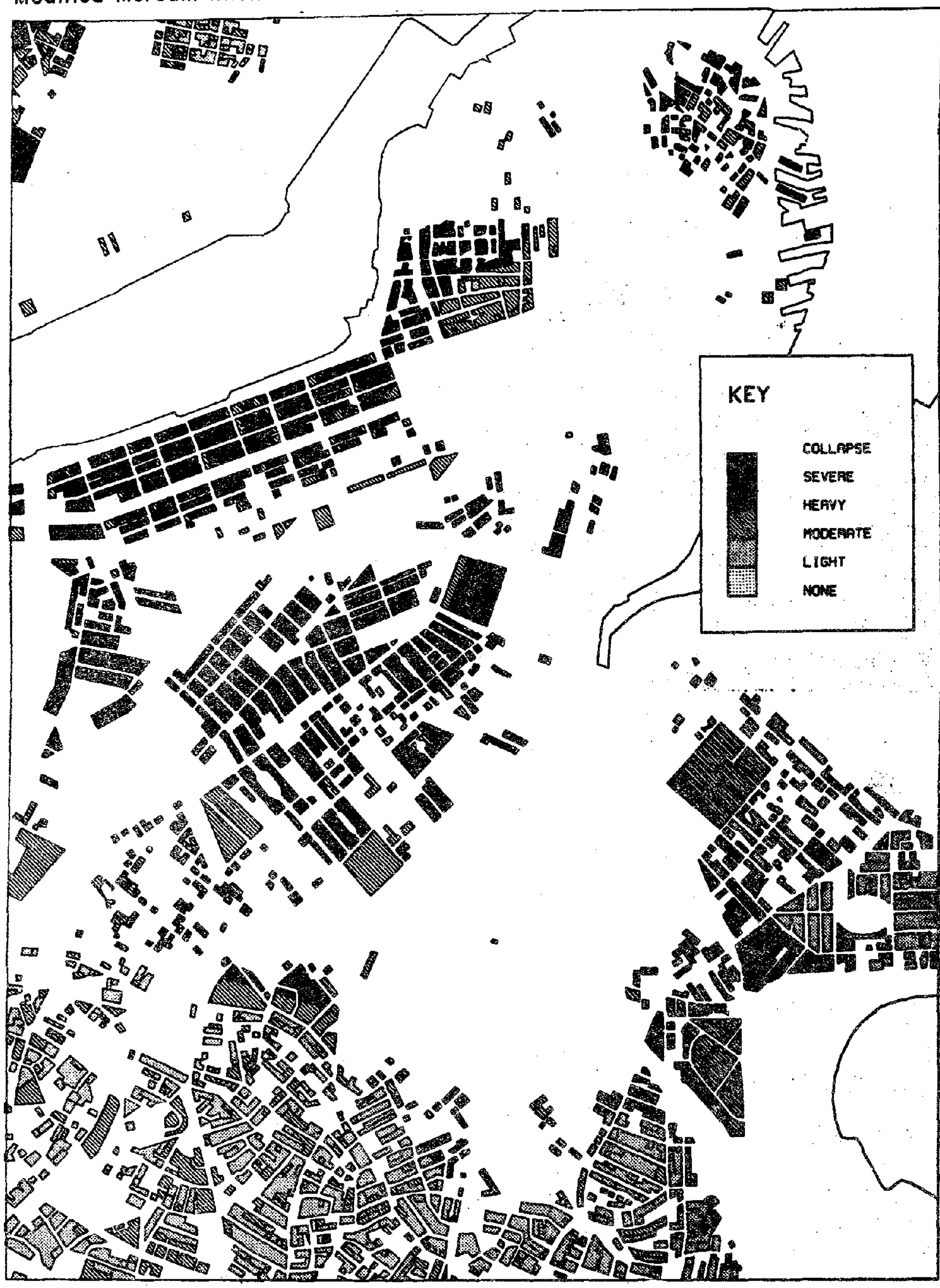


Figure 3. Distribution of Probable Damage States for MMI = VIII. (Areas Delineated Show Zones of Housing Expected to Experience Similar Mean Damage States. Within Any Area, a Spread of Damage Levels Can Be Expected Actually to Occur).

would benefit most by preventive policy measures. Exactly how this is done depends largely on the policy approach adopted -- a topic beyond the scope of this paper. The general notion of targeting measures, however, is generally applicable. In this context it might be useful to utilize generalized zone maps of the type indicated in Figure 4 as a basis for establishing different regulatory areas within a city which could be linked to different different rehabilitation requirements. Clearly for such zones to serve as regulatory districts, more care would have to go into delineation of zones.

CONCLUSION

Throughout this paper it has been argued that there is great value to looking at the seismic performance of aggregate groups of structures. This way of thinking, however, is relatively unexplored -- as is evidenced by the sparseness of data defining the seismic performance of families of buildings -- and could be considerably sharpenned up in terms of both method and the data that is input.

The approach also requires the development of methods of manipulating and analyzing data having a geographic basis -- a subject area traditionally of marginal concern to architects and engineers concerned with buildings. The use of a computer-based cartographic-data base, for example, was an absolutely essential ingredient in the study.

The idea of targeting policy response measures on the basis of studies of aggregate groups of buildings is felt to have fundamental merit not only for its own sake but also in terms of putting these measures on a rational basis. Too many important policies concerning buildings have been made on the basis of limited insights into the true extent of the problem addressed. Some of the techniques advocated could be applied to other concerns as well, e.g., fire or energy.

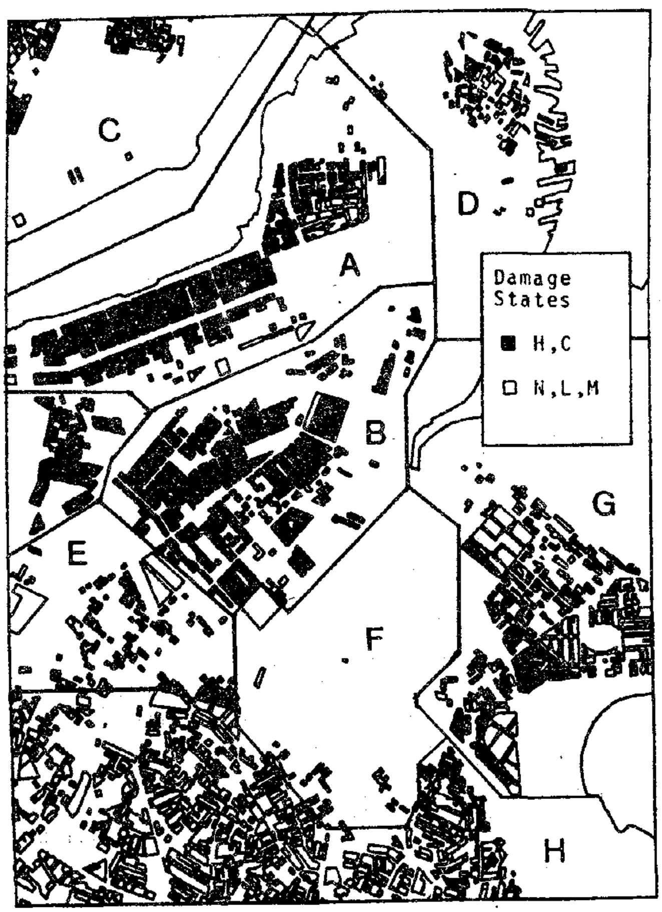


Figure 4. Target Areas. (Housing Areas are Grouped into More General Zones on the Basis of Both Expected Damage Levels and Natural Neighborhood Groupings).

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COMPOSITE STRUCTURAL ACTION FOR REALISTIC BUILDING REHABILITATION

DR. J. K. SRIDHAR RAO,
PROFESSOR OF CIVIL ENGINEERING
CALIFORNIA STATE UNIVERSITY, LONG BEACH, CA. 90840

ABSTRACT:*

Conservatism in structural design procedures used in codes with unrealistically high safety factors caused by unrealistic neglect of the actual composite interactive behavior between different structural elements and different materials, is indeed very expensive, especially for building rehabilitation and urban renewal projects in developing countries. There is a greater need for simplified analytical methods for incorporating realistic composite action so that codes could be performance oriented rather than the present prescriptive, materials oriented codes developed with different traditional practices. This should be coupled with actual field studies on prototype by load tests for a right mix of theory and practice in a realistic manner. This paper summarizes the work of 1) a reseach project on "Composite Structural Systems in Buildings" sponsored by the National Bureau of Standards (Center for Building Technology), Washington D.C. under International Grants G-91 program at IIT Kanpur and 2) Consulting work of Building Rehabilitation of a textile mill factory in Kanpur made up of lime concrete with corroded steel bars and waterproof course of brick concrete (Surkhi Concrete) by actual load tests to 1-1/2 - 2 times design live loads. Some recommendations of composite interaction of R.C. beams and masonry walls are given. Examples of "empirical design by the seat of the pants approach" for housing construction for rehabilitation of refugees in New Delhi area with proven good structural performance in the field and realistic specifications are given. These would help in developing medium to lower cost solutions in the context of the building industry in India. Experimental housing schemes with innovative design procedures and scaled down specifications would help in building rehabilitation programs together with programs for realistic evaluation of strength and serviceability of materials and structures.

*PAPER IN ABSTRACT FORM ONLY. FULL PAPER NOT SUBMITTED.

INTRODUCTION

In engineered structures, the design is done sequentially without considering the beneficial effects of composite action or taking a systems viewpoint. (Ref. 1, 3, 4, 10). This is a programmatic approach to design. After a functional design of spaces and layout is made, structural design of the frame/skeleton is made. (Even in this, composite action of three dimensional structural effect of different elements although done in monolithic Reinforced Concrete Construction, as in T-beam action, is not used in design of Timber frame construction, trusses, bracings, diaphragm . . Ref. 7). Then the design elements for sound insulation (infilled walls between frame, partitions, ceilings), thermal protection, energy conservation, supporting structural elements (secondary structural members) for utilities, service ducts, waterproofing courses, etc. are made. The beneficial effects of composite action if there is sufficient shear connection between elements is often neglected. This builds up a lot of conservatism in design and sometimes the secondary structural members may help in primary structural action if there are some deficiencies. This was observed in the structural performance evaluation of a Prestressed Concrete Folded Plate Structure on tall vertical columns with design concept weakness in the function of end diaphragms of a V-shape folded plate, wherein the structural safety was enhanced by lateral service duct members of R.C. (which was considered to be secondary structural members). In another structure which was an old textile mill building built in the 1920's with Rolled Steel Joists, Reinforced Lime Concrete the corrosion of steel as expected over a period of time, did not actually cause undue structural distress (as is normally expected in design) because of the presence of 6" course of brickbat concrete with lime and powdered bricks (Surkhi Concrete) thereby providing a shallow arch action. Load tests conducted by the author indicated that it could take 50 psf without undue cracking. The structural rehabilitation required 1" cement concrete guniting with provision of steel bars or use of corrugated steel as form and fireproof coating; and/or limitation of roof access to prevent overloading by storage and proper precaution to prevent flooding. Thus realistic testing together with composite action helped in economic rehabilitation of an industrial building in Kanpur, U.P.

On the other hand, non-engineered buildings with brick masonry and other construction features could use the results of structural experimental results and finite element methods to properly design for optimum specifications of brick and mortar for structural purposes while keeping in mind the construction requirements for shrinkage and temperature control. (Ref. 5 and 9). In the area of highway and airport pavement evaluation for repair and rehabilitation the need to use composite action of pavement layers of materials with soils rather than the usual rigid/flexible pavement classification, based on actual performance could save lot of money. (Ref. 8). Load tests and field tests play an important part in cost-effect rehabilitation. Another interesting use of non-engineered construction is the traditional housing design based on conceptual knowledge of experienced housing contractors/masons in New Delhi in post-war housing on lots of 24' x 50', in series, wherein R.C. columns 9" x 9", 4½" x 13½" with footings, beams and slabs are integrated three dimensionally with 4½" brick masonry walls and grade beams to provide composite action to effectively carry the loads in service. Traditional structural design methods which neglect composite action may brand such structures as unsafe unless it is proof-tested (non-destructive) or analysed using finite element methods. (Ref. 5). In terms of static loads, design using $\frac{WL^2}{8}$ for maximum bending moment is conservative considering conservative estimates of $\frac{WL^2}{8}$ for R.C. Beams will be infilled brickwalls (sundried bricks) with openings and under $\frac{WL^2}{100}$ for case without openings based on tests at IIT Kanpur.

SEISMIC HAZARDS MITIGATION: Towards "Systemic Integration" and Need to Take Composite Action into Account

Earthquake resistant design of structures using building codes is such a familiar activity in design practice, at least in earthquake areas that few people yet realize the highly divided character of our knowledge of it at the level of professional competence, in terms of real understanding of limitations of oversimplified quasic static pseudo elastic calculation methods which neglect composite action at both operative and design levels, or give much thought to the improvements which can and must be made. Composite action with architectural veneers, ceilings, partitions, masonry infilling, secondary structural members (such as supports for utility and service ducts) connected with primary structural members stiffen up a structure so that in some cases the structure is safe at operational loads inspite of some design errors (within limits), if there is proper shear connection between them. However, in dynamic range caused by severe design level earthquakes the damage to filling materials must be kept within limits, and the resulting flexibility may indeed reduce dynamic forces in the primary structural members in terms of life safety.

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PREDICTING WIND DAMAGE TO BUILDINGS

Kishore C. Mehta **James R. Mc Donald**

Institute for Disaster Research
Texas Tech University
Lubbock, Texas

Experience gained from more than fifty post-storm investigations of damaged buildings has helped to develop procedures to predict wind damage to buildings, should they experience cyclone, hurricane, or other types of windstorms. Brief descriptions of two procedures that involve different levels of engineering effort are presented in this paper. Use of the two procedures provides architects and engineers with information on potential building damage in windstorms. This information could be used to retrofit a building and thus make it more windstorm resistant. A case study of a school building damaged by a tornado is presented in the paper to illustrate the capability and potential of the procedure.

INTRODUCTION

Rehabilitation of buildings has become popular in recent years because of the high cost of new construction. In planning rehabilitation, retrofitting and upgrading the building to mitigate the effects of natural hazards should be considered. One natural hazard that is prevalent throughout the world is windstorms. If a building's architectural and structural systems are upgraded to resist wind effects, it will reduce injuries and deaths and will mitigate property losses. Retrofitting of a building must be done on an individual basis. However, organized procedures for predicting wind damage to a building are useful in planning and accomplishing a retrofit project.

In post-storm investigations conducted by the Institute for Disaster Research at Texas Tech University, the type of damage to various classes of buildings has been systematically categorized (see Minor and Mehta [1979]). The knowledge gained from these post-storm investigations is utilized in the development of procedures to predict potential wind damage to existing buildings. The procedures are described in a paper by Mehta, et al. [1981]. The procedures are briefly presented in this paper. In addition, a case study of damage to a school building by a tornado is presented. The case study illustrates the capability and potential of the prediction procedure.

WIND DAMAGE PREDICTION PROCEDURES

Two procedures that involve different levels of engineering effort are developed. Level I is a subjective approach, whereas Level II is an analytical approach. The Level I procedure is accomplished by an on-site survey of the building. The Level II procedure requires structural analysis of the components of the building. Synopses of the procedures are presented below; details of the procedures are found in a paper by Mehta, et al. [1981] and reports by McDonald and Lea [1978] and Smith, et al. [1979].

Level I Procedure

The subjective procedure is relatively simple and can be performed by professionals who are familiar with wind-caused building damage. The damage potential is determined by subjective judgment based on a survey of the building and completion of a questionnaire. The complete text of the questionnaire is given in the report by McDonald and Lea [1978]. The questionnaire is completed while conducting the following specific surveys and studies:

1. Survey of exterior of the building
2. Survey from the roof of the building
3. Survey of mechanical and electrical equipment
4. Study of drawings and specifications

Features observed in the survey are compared with the performance of similar features in buildings that have been affected by windstorms. Three principal types of potential damage are: (1) overall structural collapse; (2) failure of individual structural or architectural components; and (3) breach of building containment. From responses on the questionnaire and from observation of building features, subjective judgments on potential damage to the building can be made.

Level II Procedure

The analytical procedure not only predicts the type of damage, but also wind speeds at which a sequence of failure will take place. The procedure is divided into three tasks: (1) data collection, (2) analysis, and (3) interpretation. Each task consists of three increments: (1) data, (2) function, and (3) results. This three-task/three-increment matrix is shown in Table I. The results of each task are the input data for the next task. This organization of the procedure premits appropriate accomplishment of the tasks and leads to final results that provide damage sequence and associated wind speed ranges. The procedure assumes that the building may experience wind from any direction and that each building component experiences wind from the most critical direction.

TABLE I

INCREMENTS OF THE THREE-TASK ANALYTICAL PROCEDURE

Task	Data	Function	Result
1. Data Collection	Collection of: °structural drawings °architectural drawings °specifications	Action of: °site visit °determination of material strengths	Knowledge of: °structural systems °material strengths °site characteristics
2. Analysis	Knowledge of: °structural systems °material strengths °site characteristics	Analysis using: °structural mechanics °wind loads °engineering judgment	Establishment of: °threshold-failure wind speeds for each building component
3. Interpretation	Knowledge of: °threshold failure wind speeds for each building component	Analysis using: °structural response °engineering judgment	Establishment of: °damage sequence °wind speed ranges °probability of damage

The structural response of a building component or frame is made up of a static and dynamic part. For low-rise buildings and relatively stiff components, the contribution of the dynamic part of the response can be neglected. The fundamental frequencies of low-rise building frames or building components are much higher than wind gust frequencies. For example, components such as masonry walls or metal roof decks have fundamental frequencies greater than 3 Hz, while most of the free field wind gust spectrum energy, measured by Kim [1979], is in the frequency range that is less than 0.5 Hz. The disparity between fundamental frequencies of building components and gust frequencies of the wind suggests that the dynamic part of the response is negligible for low-rise building frames and most building components.

The analytical procedure requires determination of threshold wind speeds for each postulated failure mode of components and frames. The threshold wind speeds are used to formulate a damage sequence which defines the damage to a building for increasing values of wind speeds. Engineering judgment is essential to define progressive failure of the building.

If it is necessary to establish probability associated with a predicted damage sequence, a wind speed hazard probability model is required. McDonald [1980] has developed a method to determine tornado hazard at specific sites in the United States. A similar approach could be used to determine wind hazard at any site in the world, providing the wind speed records are available.

CASE STUDY

Results of analyses performed on a damaged school building are presented to illustrate that such calculations are applicable to existing structures. The school building was damaged by a tornado in Wichita Falls, Texas on April 10, 1979. Following the tornado, the damage was documented by a team of researchers from Texas Tech University. In addition to photographing the damage, specifications and construction drawings for the building were obtained. The tornado moved from southwest to northeast, thus the initial impact of the tornadic winds was on the southwest corner of the building. Because of the rotational nature of winds in tornadoes, the building experienced winds from several different directions. A brief description of the building and the tornado damage are presented below along with the postulated damage sequence.

Building Layout - The one-story building is rectangular in plan with overall dimensions of 345 ft x 430 ft (105 m x 131 m). The height of various levels of the flat roof range from 14 ft (4.3 m) to 20 ft (6.1 m) above grade. In an area over the gymnasium the gabled roof peak is 29 ft (8.8 m) high. The unusual structural system consists of cast-in-place reinforced concrete columns and wide flange steel beams. Metal deck is supported by open web steel joists that span between the steel beams. A general structural layout in Figure 1 shows the location of columns, beams and expansion joints. The concrete columns are 12 in. x 12 in. (305 x 305 mm) reinforced with four #7 bars. The columns are anchored to footing by 21-in. long (533 mm) dowels. The wide flange steel beams have nominal depths that vary from 14 in. (356 mm) to 18 in. (457 mm), depending upon the span. The beams are anchored to the top of the columns by two three-quarter in. (19 mm) diameter bolts. Resistance to lateral loads is provided by diaphragm action of the roof and the beam-to-column connections. The roof system consists of 28-gage (0.4 mm) corrugated metal deck, four inches (102 mm) of insulating concrete and a built-up tar and gravel roof. The metal deck is anchored to the open web joists in a standard way by means of puddle welds. The wall structure consists of 4-in. (102 mm) deep metal studs with gypsum wall board sheathing on both sides. A 4-in. (102 mm) nonloadbearing brick veneer completes the wall structure. Further, architectural treatment of the wall is provided by synthetic sheathing which is not intended to contribute to the structural resistance of the building. There are no windows in the exterior walls. However, all four walls facing the open courtyard (see Fig. 1) contain large glass panels.

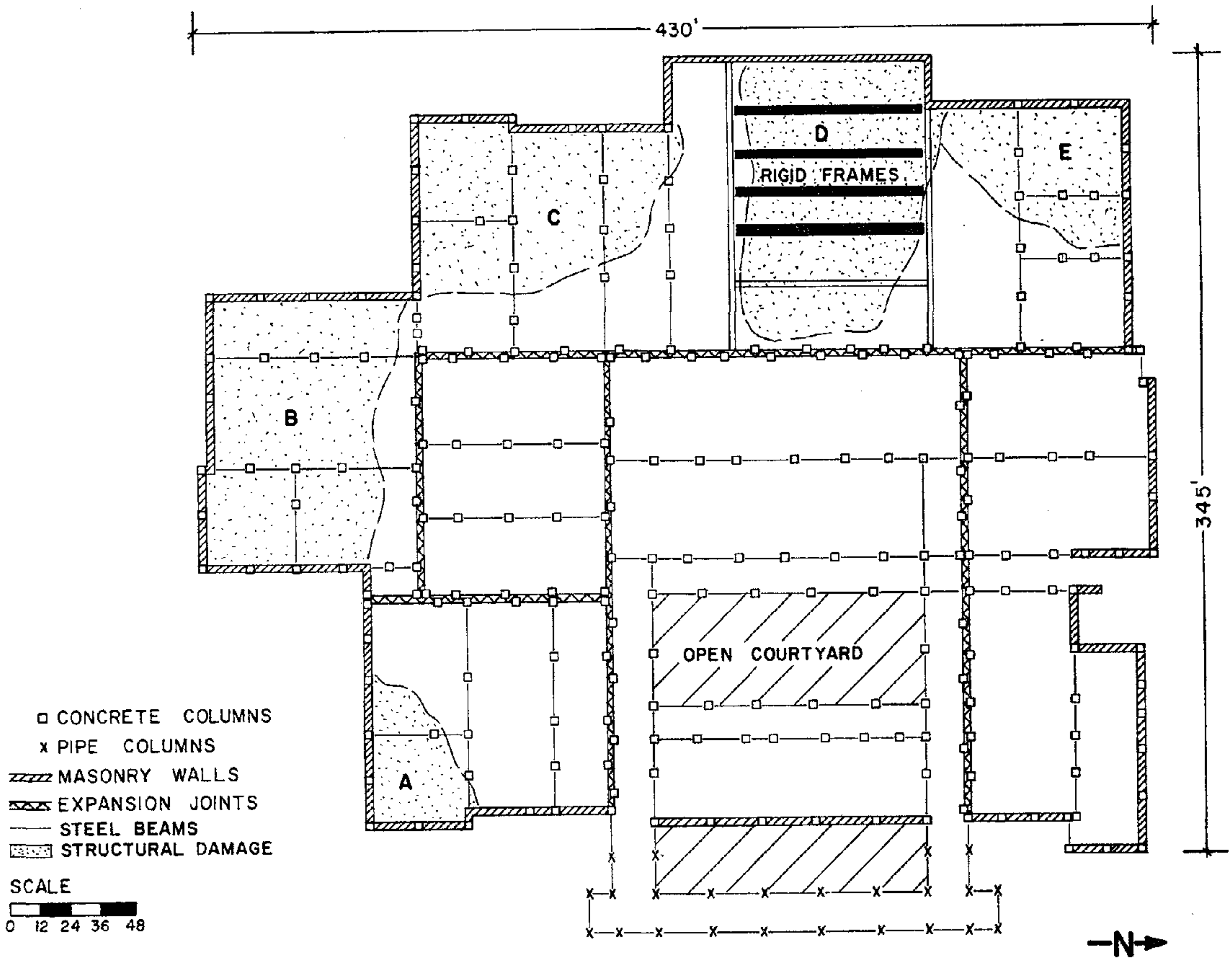

FIGURE 1. LAYOUT OF THE DAMAGED SCHOOL

The structural system in the gymnasium area consists of four pre-engineered gabled rigid steel frames spanning 88 ft (26.8 m). The roof structure consists of cold-formed purlins, gypsum roof planks and a tar and gravel roof membrane. The exterior walls of the gymnasium are constructed of 8-inch (203 mm) concrete masonry blocks with no reinforcing in the cells.

Damage - The building sustained severe structural damage. Figure 2 shows an aerial view of the damaged building. Areas of structural damage are delineated in Figure 1. Figures 3, 4 and 5 show typical damage to the building. A total of twenty-one concrete columns collapsed in areas A, B and E. The collapsed steel frames are located in area D (see Figure 1). In those areas where the columns and rigid frames collapsed, the roof structure also was removed. In area C, where the exterior walls collapsed but the columns did not, the roof deck was removed from its attachment to the steel joists (see Figure 3).

FIGURE 2. AERIAL VIEW OF THE DAMAGED BUILDING

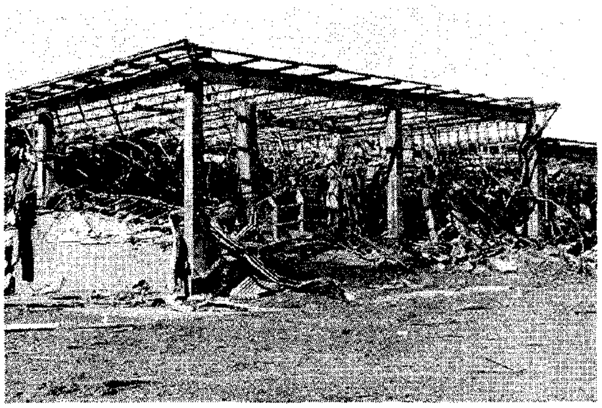

FIGURE 3. VIEW OF THE DAMAGE IN AREA C LOOKING NORTHEAST

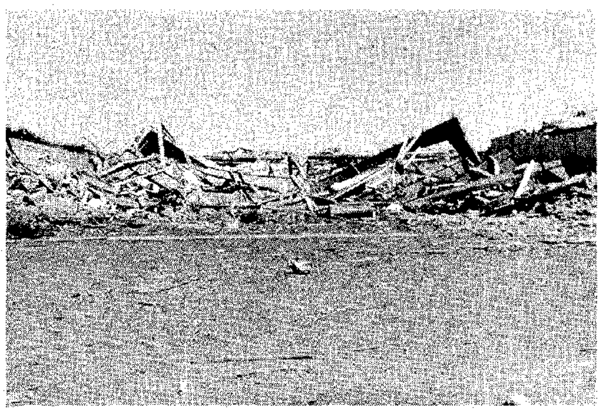

FIGURE 4. VIEW OF THE DAMAGE IN AREA D LOOKING EAST

FIGURE 5. VIEW OF THE DAMAGE IN AREA E LOOKING EAST

Roofing material over the entire building was damaged, although metal roof deck was removed only in those areas delineated in Figure 1. Glass surrounding the open courtyard and a portion of the exterior wall on the north side also were damaged. Most of the interior walls in the areas where no structural damage was evident remained intact. There was no indication of damage to the structure due to racking. The damage causing mechanism was wind pressure and not pressure due to atmospheric pressure change.

Analysis and Prediction of Damage - Documentation of damage showed that in some areas, the concrete columns collapsed (areas B and D) while in the other areas the columns did not collapse (area C). Analysis of the structural system indicates that if exterior walls are unable to resist the wind pressures and they collapse, the lateral pressures transmitted to beams and columns of the framing system are relieved. On the other hand, if the walls are strong enough to transmit lateral pressure to the framing system, the columns may collapse. Evaluation of each structural component provides threshold wind speeds at which specific damage may have occurred.

The probable sequence of damage, based on the analytical procedure (Level II) is shown in Table II. Although these calculations were made after the tornado occurred, the results likely would have been the same if they had been performed prior to the tornado. Information from such an analysis would have been useful in strengthening the building to make it more windstorm resistant or in identifying areas of occupant safety. The procedures are proving to be reliable and have a wide variety of potential application.

TABLE II

SEQUENCE OF DAMAGE

Threshold Gust Wind Speed mph (m/s)	Damage
40-64 (18-29)	Collapse of exterior wall and subsequent collapse of rigid frames in Area D.
75 (34)	Collapse of corner column in Area E.
90 (40)	Collapse of exterior wall in Area C.
100 (45) est.	Nonstructural damage; uplift of roofing material and breakage of window glass in courtyard area.
135 (60)	Collapse of columns in Area B.
145 (65)	Uplift of metal roof deck in Area C.

CONCLUSION

Procedures to predict wind induced damage to existing buildings are described. Level I procedure requires a small amount of effort, though it shoud only be used by architects and engineers who are experienced in assessment of wind induced building damage. Level II procedure requires a significant amount of effort; it has the potential of providing sequence of damage and probabilities of specific damages. These procedures could be used in planning for retrofit of existing buildings and thus to improve their resistance to wind loads.

ACKNOWLEDGEMENTS

The procedures to predict wind induced damage are developed based on damage documentation efforts of the personnel of the Institute for Disaster Research, Texas Tech University and research efforts of several graduate students. Support of the documentation efforts by the National Science Foundation, the U.S. Nuclear Regulatory Commission, and the National Oceanic and Atmospheric Administration are acknowledged. Support to analyze existing buildings was provided by the Veterans Administration and Argonne National Laboratory. Work performed by Pat Lea, Douglas Smith and David Spears during their graduate studies is used in this paper; their efforts also are acknowledged.

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APPLICATION OF POLYMERS FOR REHABILITATION OF HIGH-RISE CONCRETE STRUCTURES

by

Dov Kaminetzky

Feld, Kaminetzky & Cohen, P.C.
New York, N.Y.

SUMMARY

This paper describes the rehabilitation of two seven year old highrise 45-story concrete structures using polymer mortars. Time dependent creep and shrinkage imposed high stresses with resulting damage in the form of shearing and spalling of concrete slabs at a highrise apartment complex. Additional damage was caused by corroding reinforcing bars and "pop-outs" near the exposed concrete surfaces.

In-situ stress relief tests using electrical strain gages were performed at various levels and indicated a rather high level of compression stresses. A computer model was analyzed for the effects of dead and live loads coupled with long time creep and shrinkage. The correlation was sufficiently close to establish the main mechanism of damage.

A comprehensive repair program was undertaken. This included the use of polymer powered mortars. The no water mix was very easy to apply, cured and hardened fast and resulted in a very hard surface with high tensile and compressive strengths.

The separation of the cladding by introduction of "soft joints" was effected so as to reduce the repetition of development of high stresses.

Introduction

The purpose of this paper is to present the results of field, laboratory and office studies relating to the distress conditions of the exterior masonry walls and the deterioration and damage to the exposed concrete slabs at a residential towers project in Manhattan, New York.

The scope of the work included:

a. The evaluation of existing conditions and preparation of a condition survey. Field engineers conducted inspections of the structures and recorded existing cracks, spalls and other visible defects.

b. Preparation of and evaluation of laboratory and field testing programs.

c. Preparation of an Engineering Report including conclusions and recommendations as to rehabilitation of the structure.

d. Review of the actual repair of the structure and the corrective construction.

General Description of the Project

The project is located in Manhattan, New York City, U.S.A., and comprised of two high-rise apartment house concrete structures 46 stories high (see Fig. 1), connected by several low-rise concrete structures.

The high-rise structures, which are the subject of this paper, were constructed of 6½" flat plate concrete slabs supported on concrete columns, founded on footings bearing on 20 ton per square foot rock. The slabs were designed as lightweight concrete with 3500 psi strength at 28 days. The columns and shear walls were designed as 5000 psi stone concrete below the 16th floor and light weight concrete above the 16th floor. (4000 psi from 16th to 26th floor and 3500 psi above the 26th floor.)

High strength reinforcing bars of 60 ksi yield were used for main tensile reinforcing. There were no dropped spandrel beams. Exposed face of slabs is shown on Fig. 2.

The structures won Architectural Design Awards for Excellence in exposed concrete structures.

In November 1979, FKC was requested to study the existing conditions of spalled concrete from the spandrel faces and the spalled brick just below the slabs.

The first FKC action was a recommendation to remove loose brick and concrete chips in order to minimize hazards related to the possibility of falling brick and concrete pieces.

Later, five concrete samples were sent to the laboratory for petrographic and chemical testing. Concurrently, stress and strain relief tests were conducted near the 6th floor, the 19th floor, and at the 38th floor. The specific location of these tests was selected because

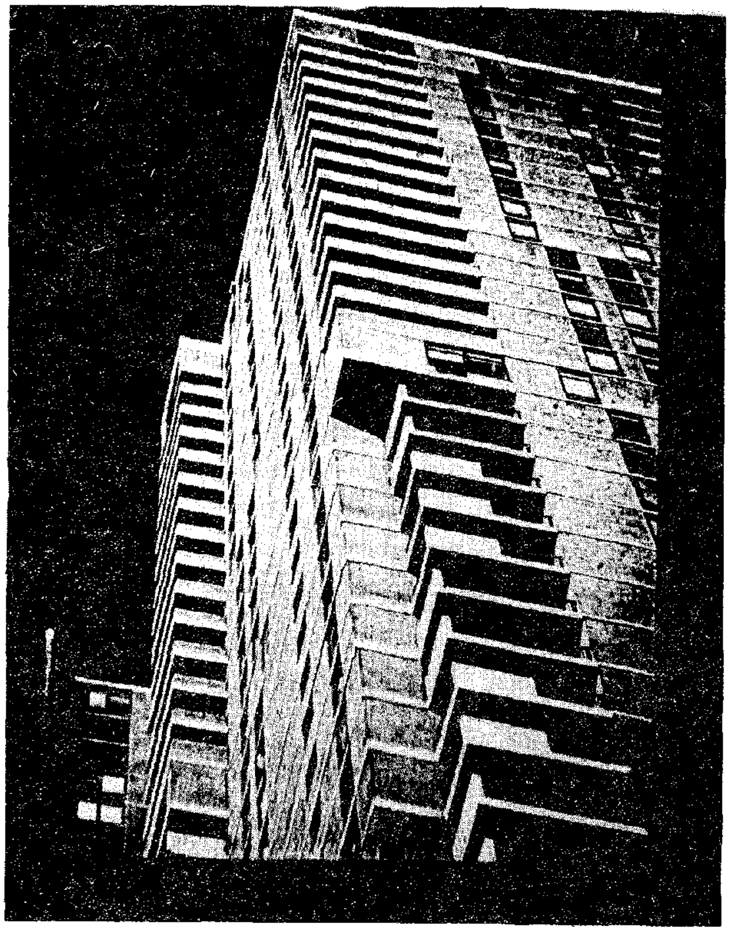
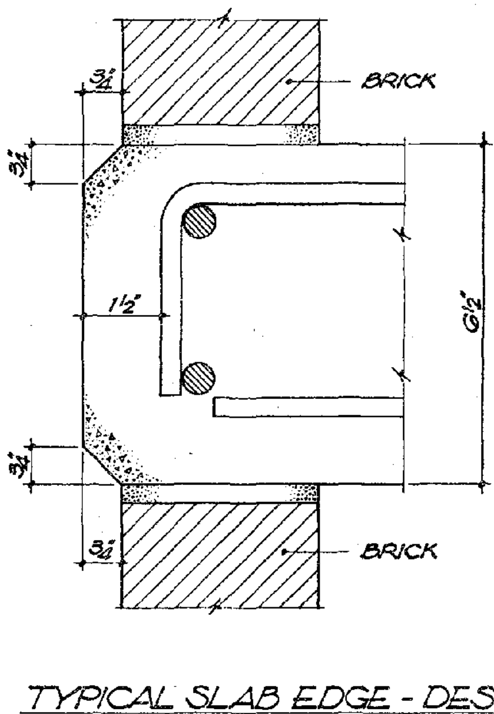


FIG. 1 - GENERAL VIEW OF PROJECT



TYPICAL SLAB EDGE - DESIGN DETAIL

FIG. 2

spalling of concrete and masonry was evident there at various levels. Seven brick samples were removed from the area of the stress-relief tests to Columbia University for laboratory tests. The results of these tests are included and discussed in the following sections.

During the intervening days condition surveys were taken of various distress locations. Also concrete cover on reinforcing bars and the actual slope of the concrete slab face were recorded.

Later, nine (9) exploratory openings were opened at various levels of Building "A". The purpose of these explorations was to gain information on the existing conditions of the slab and wall construction, their relative locations and connection details.

Discussion

Description of Damage

A complete survey of the damage was conducted by visual observation and recorded. Damage was identified in the following categories:

1. Exposed reinforcing bars (see Fig. 3).
2. Spalled concrete (see Fig. 4).
3. Spalled concrete patches.
4. Chipped brick units.
5. Cracks in concrete and brick units (see Fig. 5).

The predominant damage is in categories (1), (2), and (4). The concrete damage was to the face of the exposed slabs and the damage to the brick was at the top course, just below the bottom of the exposed slabs.

Possible Causes for the Distress

The following main causes have been considered and studied:

a. Spalling and cracking of brick and concrete because of freeze-thaw cycles.

b. Spalling of concrete face because of corrosion of reinforcing bars.

c. Spalling and crushing of brick and concrete due to vertical compression stresses as a result of dead, shrinkage and creep loads, separately and in combinations.

(a) Freeze-Thaw Cycle

Hardened dry concrete is not damaged by freezing and thawing; hardened wet concrete may be so damaged unless suitable measures are taken to prevent it. Table 3.1 gives results by Collins which illustrate this point.

TABLE 3.1—DRY CONCRETE IS NOT DAMAGED BY FREEZING AND THAWING

Number of cycles of freezing and thawing	Compressive strength as percent of strength after 7 days of moist curing	
	Frozen wet	Frozen dry
0	100	100
10	141	165
20	137	189
30	119	201
40	99	211
50	63	220
60	0	228

The cause of deterioration of concrete in freezing and thawing is the freezing of water therein. Concrete includes void space capable of containing water that can freeze. Stresses accompanying the change from water to ice may cause deterioration either of the hardened paste or of the aggregate, or both. The actual mechanisms by which damage is done is quite complex.

"The inevitably present empty space in concrete is not sufficiently near to all the capillaries in which ice may form to be of appreciable aid in limiting the resulting stresses to a safe value. Although the required space exists in the concrete, it is not available to the water or ice. Experience shows that it is necessary to have some empty space within about 0.005" of every point in the hardened paste to be fully effective in this regard. This requirement can easily be met by the use of entrained air, which thus provides a fundamental solution to the problem of resistance of the cement paste part of hardened concrete to freezing and thawing." (ACI Mono. #4.)

The samples of concrete taken from the structures were found to contain between 5½ to 6 percent air content, which is an acceptable value. The petrographic testing ruled out freezing damage as a possibility based on the samples tested.

(b) Spalling As a Result of Corrosion of Reinforcing Bars

There is sufficient evidence to indicate that corroded reinforcing bars were found in areas of spalls and damage. The process is that of corrosion of steel in the presence of water and air with resulting iron-oxides. These oxides expand in formation and the resulting stresses are sufficiently high to push outward (path of least resistance) and crack the concrete face.

The petrographic testing confirmed that this mechanism of failure of the concrete occurred at the Manhattan structures.

(c) Damage Due to Vertical Compression

The main vertical structural elements for support of vertical load in the structure are the reinforced concrete columns. However, there are two other vertical members which are not classified as "structural", but they act as such. These are the vertical walls; in particular, the exterior brick and block walls. Because of their great cross-sectional area and continuity, they, in effect, participate in the load transfer despite the fact that they have not so been

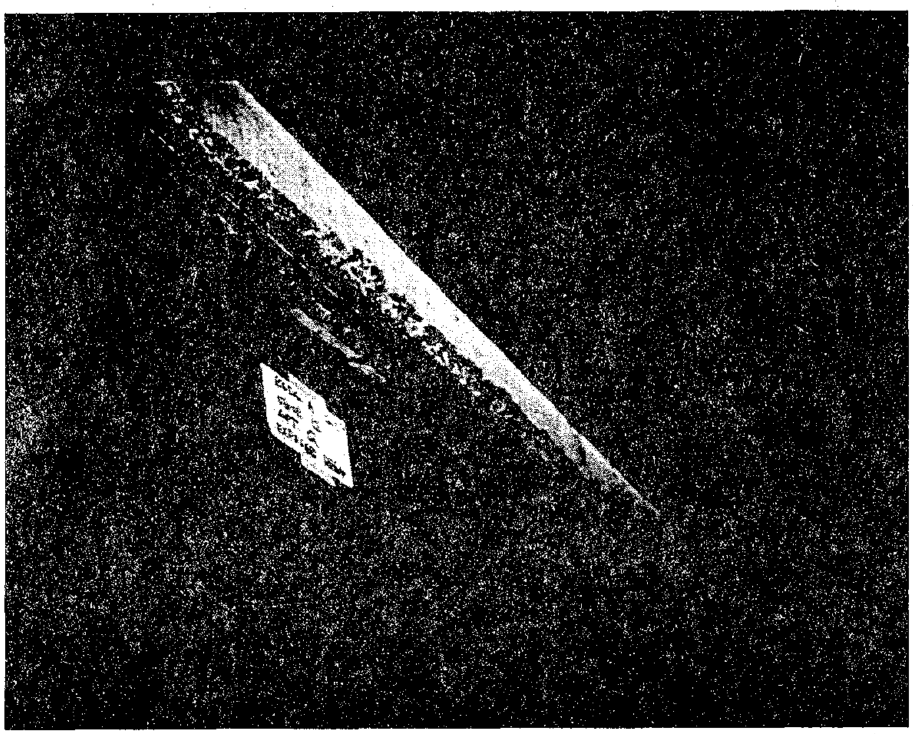


FIG. 3 - SHOWING EXPOSED REINFORCING BARS

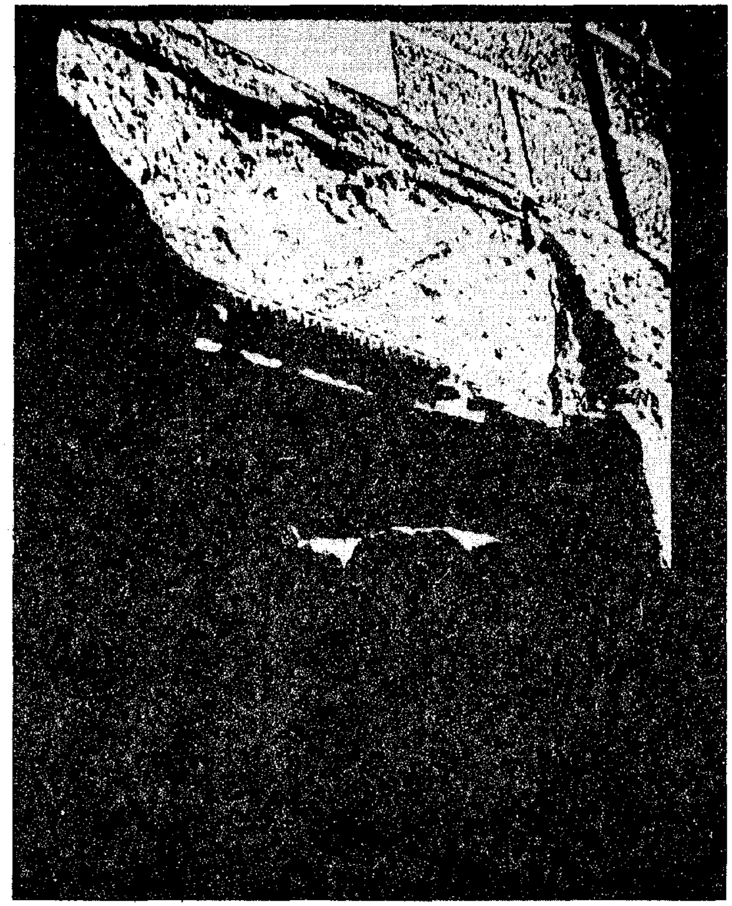


FIG. 4 - SHOWING SPALLED CONCRETE

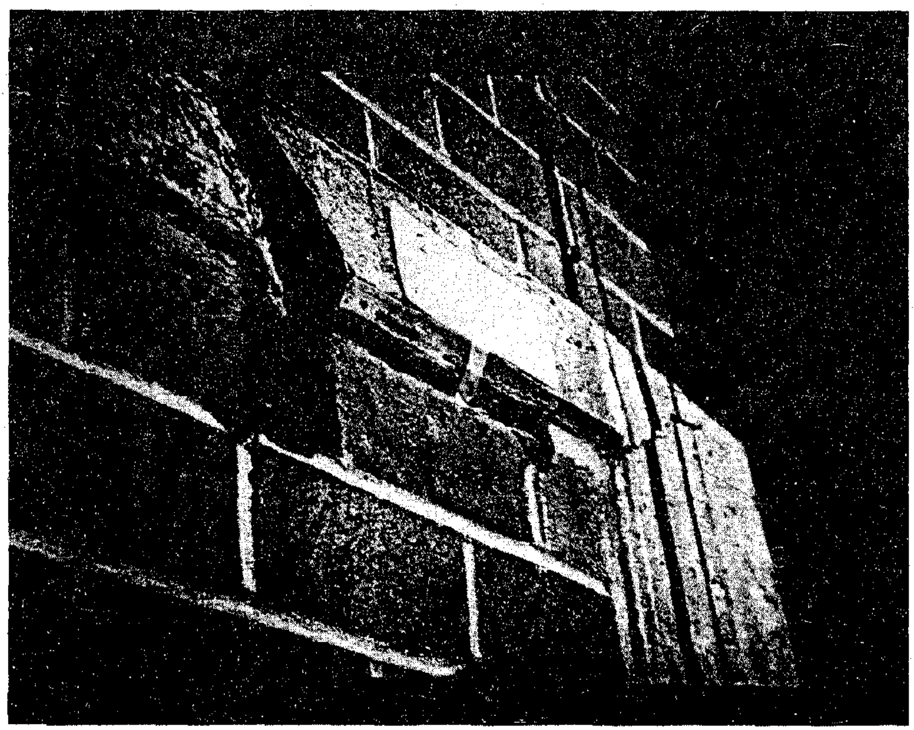


FIG. 5 - SHOWING CRACKED CONCRETE

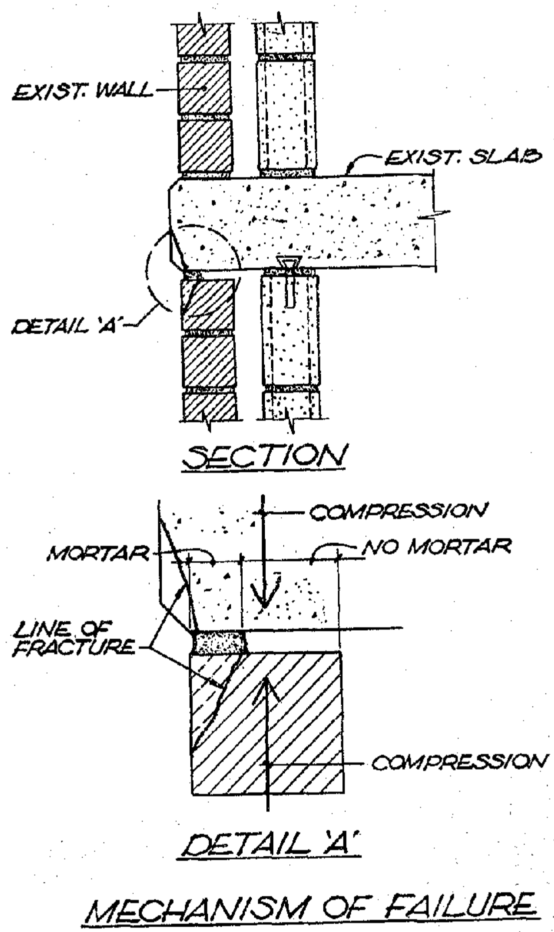


FIG. 6

designed.

It was an accepted engineering theory that where exposed slabs are used, the possibility for cumulative stresses in the brick is greatly reduced by the interruption of slabs at each level. The present New York City Code does not specifically recognize the fact of possible compression stresses in these cases.

The field tests conducted on this project combined with analytical computations clearly demonstrated the existence of these high compression stresses. This situation was magnified here because of the relatively hard brick unit. ($E = 5.9 \times 10^6$ psi.) Where a combination of high shrinkage and creep of concrete and high modulus of elasticity of brick exists, high compression stresses will result. These high stresses may cause crushing of concrete at edges (non-confined areas) and spalling of brick.

The level of the stresses in the brick had increased by the virtue of the fact that the mortar joints in many cases were not full below the slabs resulting in high compression at the brick face. (See Fig. 6.)

Brick Masonry Units

The brick units were specified in the project specifications as ASTM C-216 Grade SW Type F.B.S.

Average laboratory-tested values were found to be:

10440 psi - Compressive Strength
.65 - Saturation Coefficient
5.9×10^6 psi - Comp. Modulus (E)
2.3×10^{-6} in/in/^oF - Coeff. of Thermal Expansion

All these values are within acceptable limits. The compressive strength is quite high and the .65 saturation coefficient is a good indication of a durable brick. Therefore, no deficiencies in the quality of the brick were indicated.

Workmanship

a. The exploratory openings revealed certain deficiencies in the masonry workmanship, in particular the partially-filled mortar joints at the top of the brick and block walls. The omission of several masonry and dovetail ties are considered unacceptable workmanship, however, they had no contribution at all to the present distress.

b. Placing of reinforcing bar without proper and sufficient concrete protection cover against corrosion is a defect that should have been avoided by extra care in the field or the use of side form spacers.

Evaluation of Stresses by Field and Analytical Methods

a. Field Studies

The field stress relief tests confirmed the existence of high stresses in the brick.

It calls for the clear conclusion that new construction of this type should include "soft joints" at each level directly below the exposed slabs. Stress relief tests at the 38th floor is shown in Fig. 7. Test results are shown in Tables #1 and #2, and the total stress relief summary is shown in Table #3.

b. Analytical Studies

Computer analysis runs were performed in order to evaluate the magnitude of the stresses due to dead loads, shrinkage and creep and their effect on the brick facing in the block back-up. A typical model and schematic of the runs are shown in Fig. 9.

Run #003 was performed using the elastic modulus for the brick of 3×10^6 psi, which is an average value for brick.

Run #005 was identical to Run #003 except that a value of 5.9×10^6 psi was used for the modulus of elasticity for the brick. This high value is the one obtained by the laboratory from sample brick units taken from the wall at the project.

Run #004 was identical to Run #003 with the exception that the block back-up walls were omitted, based on the assumption they have shrunk and there is no positive contact between the top of the block wall and the soffit of the concrete slab.

Run #006 was identical to Run #005 except that at alternate floors horizontal expansion joints were assumed.

Table #4 contains the computations for creep and shrinkage coefficients. A comparison of the results of field test and the theoretical analysis are shown in Fig. 10. These results show amazingly high values of compressive stresses in the brick, with an average level of 2000 psi and maximum level of 3300 psi. In general, the following conclusions may be drawn from the analytical stress analysis:

(1) Cutting of expansion joints below the slabs at each level will eliminate stresses in the brick to a great extent. Cutting of joints at alternate floors will reduce the stresses in the brick, however, substantial stress will still be "locked-in" at the continuous floors.

(2) The effect of the high modulus of the brick on the level of stresses in the brick facing is considerable as may be determined by comparisons of Run #003 and Run #005.

(3) The compression stresses in the brick will be particularly higher at those floors where there is no contact between the top of the back-up wall and the soffit of the slabs.

(4) The compression stresses in the brick increase as you go down from the top floor to the bottom of the building, however, stresses as high as approximately 1,000 psi still exist at the top floors due to shrinkage and creep.

Existing Conditions As Determined by Exploratory Openings

Nine exploratory openings were cut at various locations of Building "A", at the 5th, 9th, 11th and 17th floors.

These areas were selected because of the convenience of the scaffold locations. While statistically the areas explored cannot be regarded as constituting optimum representative samples, we consider them to be sufficient based on visual inspections and other data relating to the two buildings. The actual number that

TABLE 1

Gage No.	Dist. From Edge	*Strain relieved 10^{-6} in/in (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	2"	0	- 10	- 10	- 10	30	25	20	10	- 60	35
2	$6^3/_4$"	0	- 10	- 10	85	120	110	105	50	235	250
3	15"	0	15	20	50	50	40	10	130	140	140
4	$22^3/_4$"	0	0	0	0	0	-30	60	75	70	70
5	$1^1/_4$"	0	- 20	- 15	- 25	20	5	5	- 5	- 40	- 30
6	7"	- 25	- 30	- 30	60	65	60	50	35	75	80
7	$1^1/_2$"	- 10	- 10	- 15	- 65	35	35	30	30	20	20
8	6"	- 30	- 35	- 40	180	180	175	175	170	160	160
9	$15^1/_4$"	40	40	35	50	40	25	20	45	45	40
10	$14^1/_2$"	180	175	180	190	190	195	195	200	190	195
11	$29^3/_4$"	60	80	110	110	110	200	250	260	275	275
12	22"	- 15	- 10	- 15	- 10	-10	-40	-25	-15	- 15	- 20
13	23"	0	0	0	0	0	0	0	0	5	5
14	$29^3/_4$"	0	0	10	0	0	0	5	5	10	10

*Negative indicates increased compression at that stage.

TEST RESULTS - STRAINS

A positive number gives the compressive strain relieved from the undisturbed state to the end of the stage indicated - i.e. (1) - (10)

TABLE 2

Gage No.	*Stress relieved psi (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	0	- 60	- 60	- 60	180	150	120	60	-350	210
2	0	- 60	- 60	500	710	650	620	300	1390	1480
3	0	90	120	300	300	240	60	770	830	830
4	0	0	0	0	0	-180	350	440	410	410
5	0	-120	- 90	-150	120	30	30	- 30	-240	-180
6	-150	-180	-180	350	380	350	300	210	440	470
7	- 60	- 60	- 90	-380	210	210	180	180	120	120
8	-180	-210	-240	1060	1060	1030	1030	1000	940	940
9	240	240	210	300	240	150	120	270	270	240
10	1060	1030	1060	1120	1120	1150	1150	1180	1120	1150
11	350	470	650	650	650	1180	1480	1530	1620	1620
12	- 90	- 60	- 90	- 60	- 60	-240	-150	- 90	- 90	-120
13	0	0	0	0	0	0	0	0	30	30
14	0	0	60	0	0	0	30	30	60	60

*Negative indicates increased compression.

Positive indicates total compression relieved from undisturbed state.

TEST RESULTS - STRESSES

TOTAL STRESS RELIEF SUMMARY

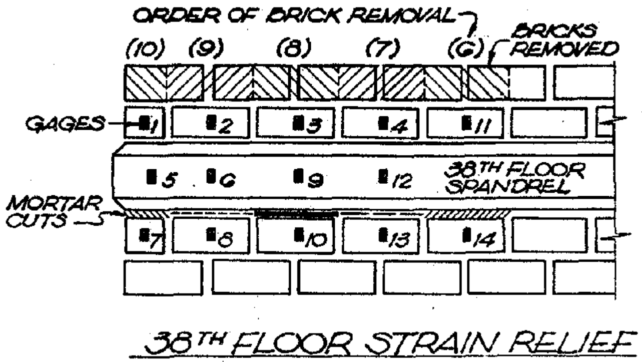
TABLE 3

STRESS IN PSI			
GAGE \ FLOOR	6TH FLOOR	19TH FLOOR	38TH FLOOR
1	440	1090	210
2	6900(max)	1180	1480
3	1060	410	830
4	710	560	410
5	320	1830	(-180)
6	470	0	470
7	1150	1890	120
8	240	1800	940
9	180	650	240
10	890	2360(max)	1150
11	-	1710	1620 (Max)
12	-	-	(-120)
13	-	-	30

TABLE 4

EFFECT OF COLUMN SHORTENING

AVERAGE TOTAL COEFFICIENT OF CREEP AND SHRINKAGE				
COLS.	36TH TO ROOF	20TH TO 36TH	FNDN - 20TH	REMARKS
W PCF	120	120	150	
f'_c psi	3,500	4,000	5,000	
$h_{avg.}$	8'-7"	8'-7"	8'-7"	
SIZE in. X in.	36 X 12	42 X 12	48 X 12	
A_S	6-#6	6-#8	6-#8	
A_T in²	459.2	549.5	603.5	$A_T = Ag + (n-1)As$
E psi	2.57×10^6	2.74×10^6	4.28×10^6	$E = 33\, W^{3/2}\, f_c'^{1/2}$
SUSTAINED LOAD KIPS	198	478	653	D.L. (SL + COL. +EXT.WALL)+ % L.L.
f_a psi	432	870	1,082	$f_a = \frac{LOAD}{A_T}$
V/S \ $\alpha^c_{V/S}$	4.5 \ 1.17	4.7 \ 1.16	4.8 \ 1.15	VOL./SURF
ε'_c X avg. \ ε'_c	0.50×10^{-6} \ 0.58×10^{-6}	0.53×10^{-6} \ 0.62×10^{-6}	0.26×10^{-6} \ 0.30×10^{-6}	$\varepsilon'_c = \alpha^c_{V/S}\, \varepsilon'_{c\ AVG.}$
ε_c	252.8×10^{-6}	535.3×10^{-6}	324.6×10^{-6}	$\varepsilon_c = \varepsilon'_c \times f_a$
ε_s	400×10^{-6}	400×10^{-6}	400×10^{-6}	ASSUMED
r	0.90	0.90	0.90	$r = \frac{\text{resid. strain}}{\text{total strain}}$
$r(\varepsilon_c + \varepsilon_s)$	587.5×10^{-6}	841.8×10^{-6}	652.1×10^{-6}	



Cases (1) - (10): Strain readings after each of the following steps --

(1) mortar cut ————

(2) mortar cut ---------

(3) mortar cut ///////

(4) mortar cut ——— —

(5) mortar cut \\\\\\\\\\\

(6) - (10) portions of brick and mortar removed as shown above

Conditions during test - hazy, 50°F

Date of test - 3/28/80

FIG. 7

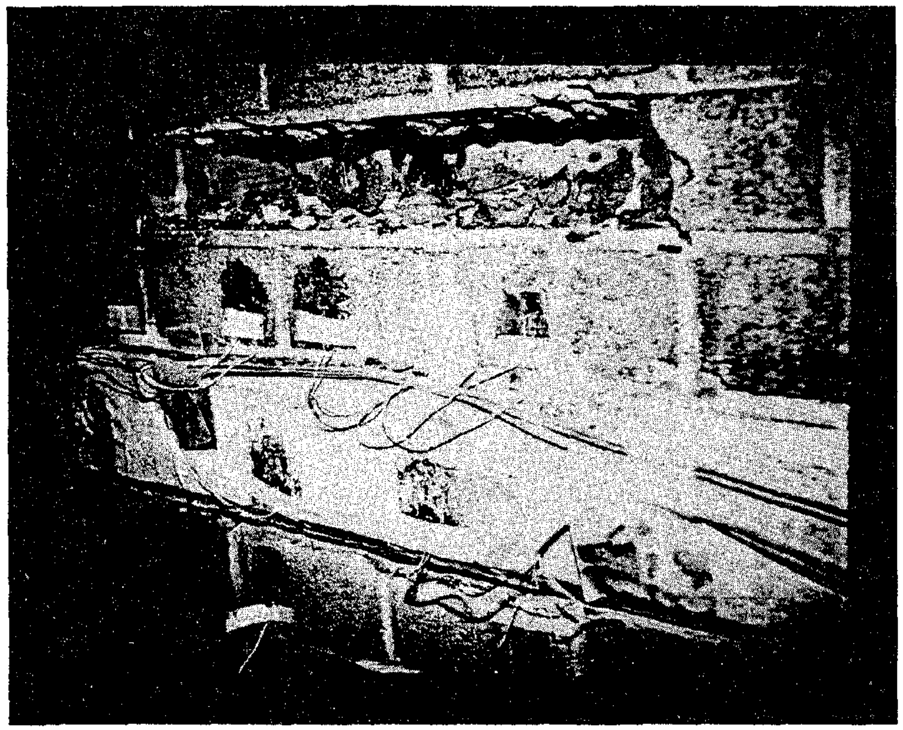


FIG. 8 - SHOWS STRESS RELIEF TEST

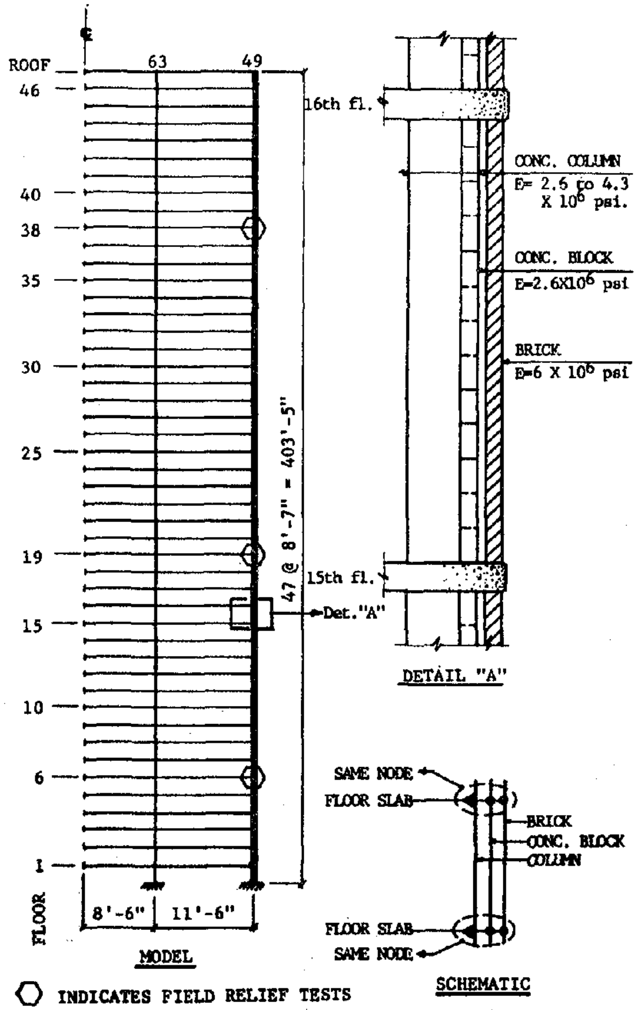

ROOF
63
49
46
40
38
35
30
25
19
15
10
6
1
FLOOR
8'-6"
11'-6"
MODEL
47 @ 8'-7" = 403'-5"
Det."A"
16th fl.
15th fl.
CONC. COLUMN
E= 2.6 to 4.3 X 10^6 psi.
CONC. BLOCK
E=2.6X10^6 psi
BRICK
E=6 X 10^6 psi
DETAIL "A"
SAME NODE
FLOOR SLAB
BRICK
CONC. BLOCK
COLUMN
FLOOR SLAB
SAME NODE
SCHEMATIC
INDICATES FIELD RELIEF TESTS

FIG. 9

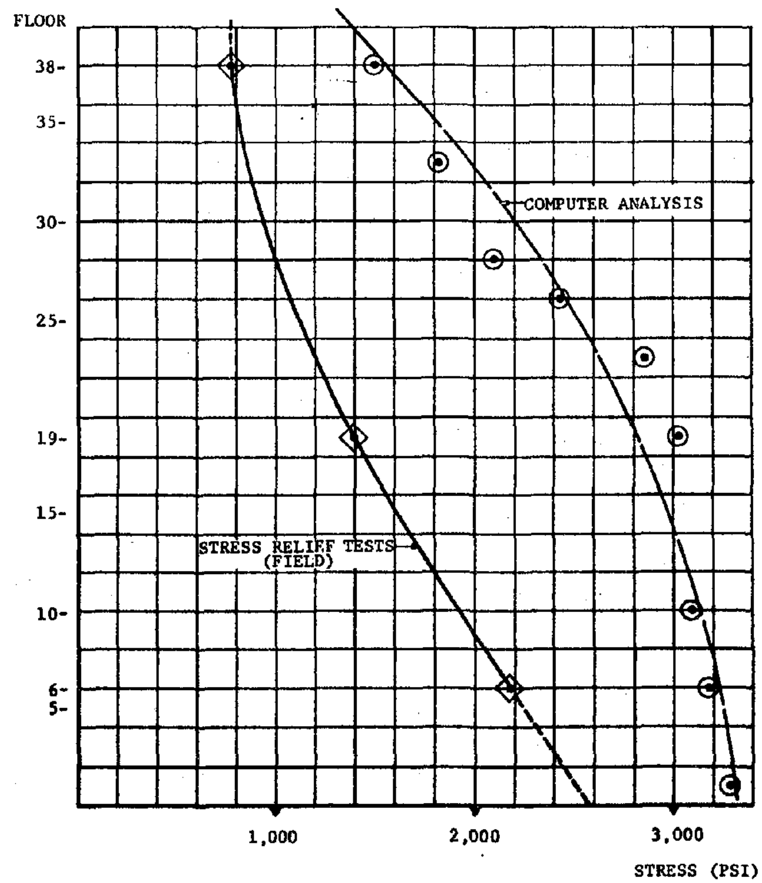

FLOOR
38
35
30
25
19
15
10
6
5
COMPUTER ANALYSIS
STRESS RELIEF TESTS (FIELD)
1,000
2,000
3,000
STRESS (PSI)

FIG. 10

will be required to determine the average quality of the workmanship will be approximately 20 for a 10 per cent maximum allowable error between the estimate to be made from the sample and the result of a full examination of all conditions of the walls.

In general, the conditions found are:

a. Most (if not all) cases of chipping of the face of the top brick course were directly related to a partial mortar joint at the bottom of the slabs. In a few cases chipping was observed with full mortar joint which may be explained by extremely high compressive stresses or non-uniform mortar.

b. Several masonry anchors tying the brick facing to the block back-up wall were missing.

c. While dovetail slots are provided in the concrete column, in most cases, dovetail masonry anchors are missing.

d. The cavity was clear of mortar.

e. Signs of compression were noticed at the top course of brick.

The General Procedure Used for Repair

1. "Soft joints" were cut in accordance with Fig. 11, which shows continuous double back-up rods and sealant. The sealant was a Special Caulking Compound "Chemcaulk 800" manufactured by Woodmont Manufacturing, with a minimum ratio of 2 to 1 (height to width). Samples of sealant were submitted for approval and color selection. "Soft joints" were installed at each level continuously all around and throughout the entire building height.

2. Two alternate methods of installation of joints were used:

   a. Removal of 1 course of brick and replacement with a shallow brick unit of the same brick as original. Color to match original brick. Samples of brick were submitted for approval of type and quality of brick (ASTM Requirements) and also for approval of color. (Used where existing joint was less than 3/8".)

   or:

   b. Cutting a joint with power saw using diamond blades. (Used where existing joint was 3/8" or larger.)

3. Mortar was matched to existing mortar in color and was the same type as originally used. Samples of hardened mortar were submitted for approval of color.

4. Broken brick units were replaced with new brick units.

5. Spalled concrete slab faces were repaired in accordance with "Procedures of concrete repair".

6. Water-repellent coating (Hydrozo) was applied to concrete slab facing (both existing and repaired).

7. Additional inspection from scaffolds was scheduled for approximately one year after completion of these repairs in order to assess their effectiveness.

Procedure for Concrete Repair

1. Loose, broken and cracked and otherwise deteriorated concrete, dirt and other bond inhibiting materials were removed and a rough, clean, sound concrete surface was provided by hammering (minimum 2 lb. hammer), chipping or sandblasting. A minimum 1½" groove at all edges was provided to avoid all feather edges.

2. All rust and corroded steel was removed from existing reinforcing bars by wire brush. Severely corroded reinforcing bars were replaced by cutting back concrete to expose sound reinforcing. New reinforcing of the same size was welded to the existing reinforcement.

3. ½" ø stainless steel pins were placed into the clean prepared concrete surface that was to be repaired.

4. After the concrete surface was prepared, repair material was applied to the concrete surface. (See Fig. 14.) Special attention was taken to assure that the bonding compound penetrated into the existing cracks.

5. Concrete was restored to original shape and geometry. (See Fig. 15.) Color of polymer mix was matched to color of existing concrete by prior testing of several mix samples. Some areas required reforming. Forms were faced with a releasing agent to prevent bond of the polymer to the form. (See Figs. 12 and 13.)

Repair Mortar

Material

Polymer modified cementitious system consists of a factory preportioned 2-component system conforming to the following requirements:

Component A is a liquid polymer emulsion of an acrylic copolymer base and additives. This acrylic copolymer has the following properties:

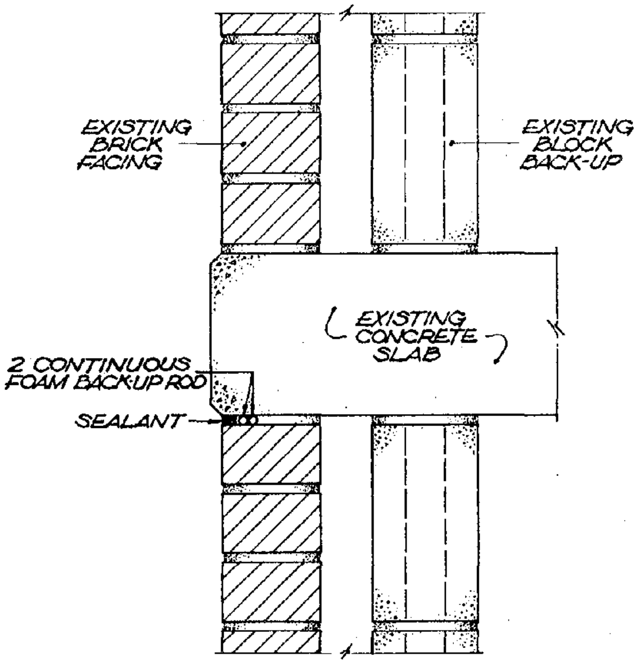
pH	4.5 - 6.5
Minimum film forming temperature	Approx. 68°F
Tear Strength	Approx. 990-1420 psi
Elongation at Break	500-900%
Particle Size Range	Less than 0.1 micron

Component B is a blend of selected Portland Cements, specially graded aggregates, organic accelerator, and admixtures for controlling setting time, water reducers for workability and a corrosion inhibitor.

The component ratio A:B is 1:7.2 by weight. The system does not contain any chlorides, nitrates, added gypsum, added lime, or high alumina cements. The system is non-combustible, either before or after cure.

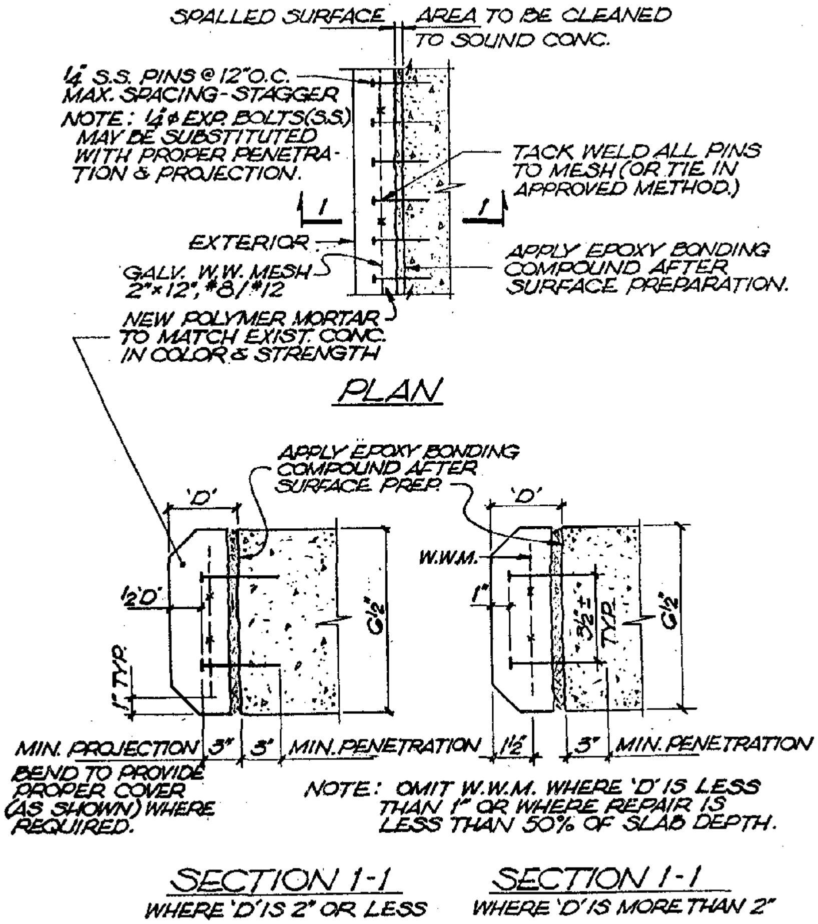
Typical Properties of Mixed Components

Application Time (Working Time)	15 min. after combining components



TYPICAL HORIZONTAL 'SOFT JOINT'

FIG. 11



REPAIR OF EXPOSED FACE OF CONCRETE SLABS

FIG. 12

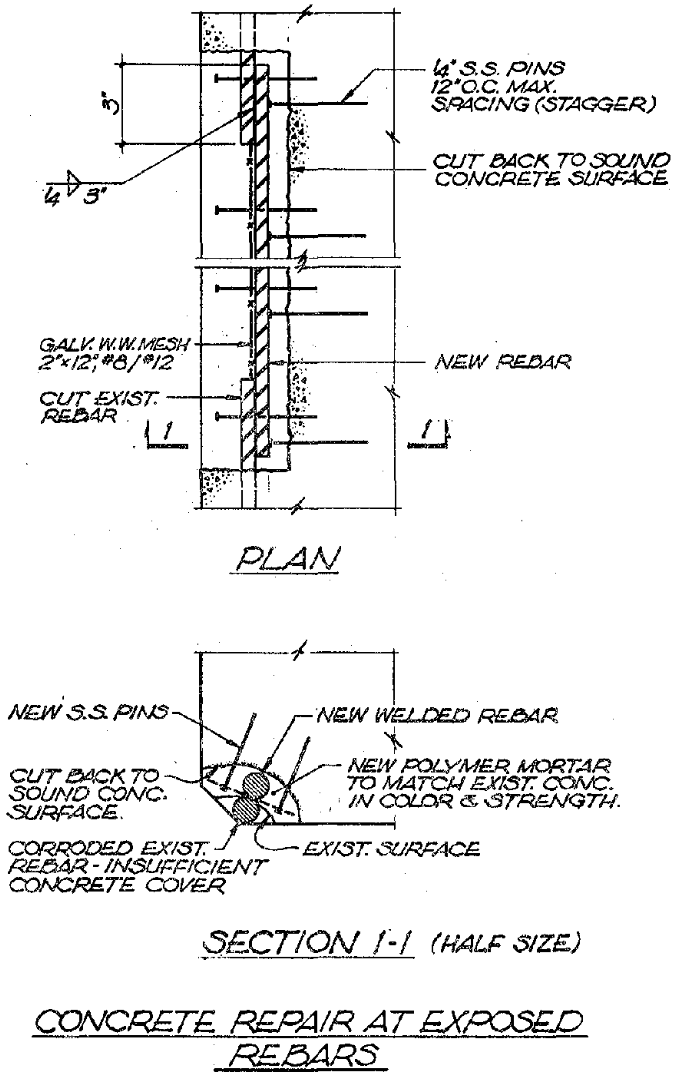


FIG. 13

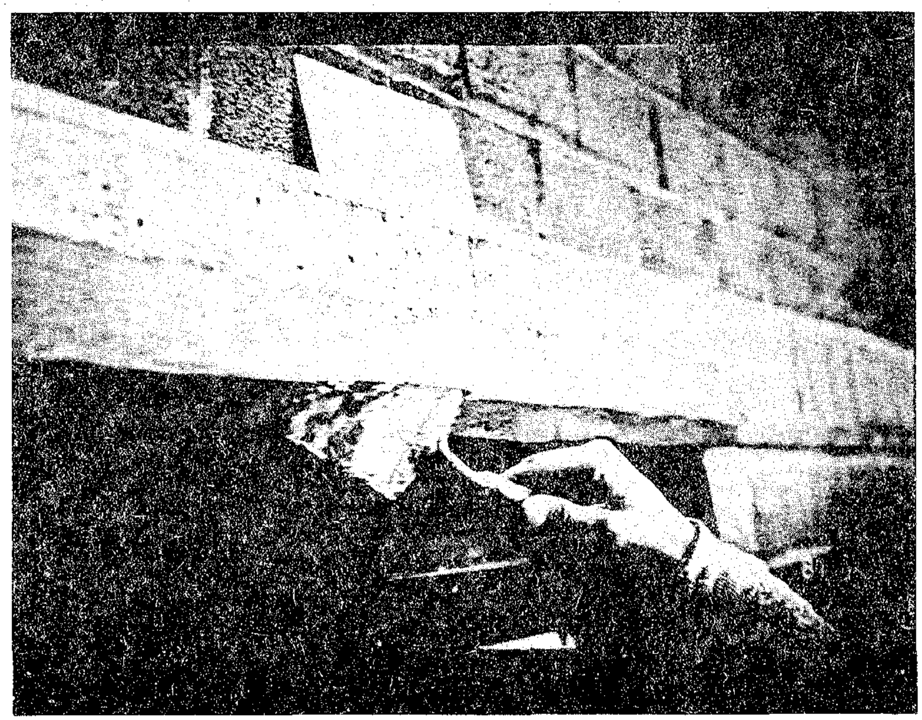


FIG. 14 - SHOWING REPAIR WITH POLYMER MORTAR APPLIED TO DAMAGED CONCRETE

Finishing Time	20-60 min. after combining components
Color	Concrete Gray

Typical Properties of Cured Material

Abrasion Resistance	6 times that of controlled concrete
Bond Strength (Pull off method)	100% concrete substrate failure
Modulus of Elasticity	4.5 X 10^6 psi
Surface Scaling (Deicing salt solution freeze/thaw)	No deterioration after 120 cycles
Compressive Strength (2 hours 50% RH)	150 minimum
Compressive Strength (28 days 50% RH)	5,500 psi minimum
(1 day)	3,000 psi minimum
Flexural Strength (28 days 50% RH)	1,300 psi minimum
(1 day)	850 psi minimum

The system is compatible with concrete.

Application

In layer 1/2" to 3/4" maximum.

Minimum substrate temperature 40°F and rising.

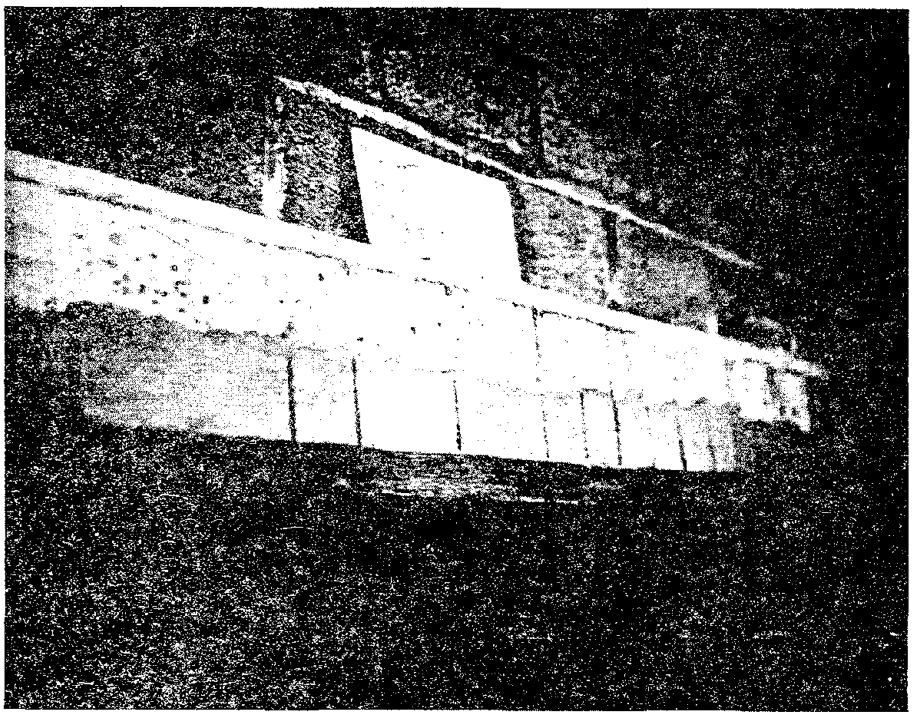


FIG. 15 - SHOWING REPAIRED CONCRETE FACE. NOTE: DARK COLOR OF REPAIR TURN LIGHT UPON CURING.

THE ROLE OF SUPERPLASTICIZED FIBER REINFORCED CONCRETE AND FIBER SHOTCRETE IN THE REHABILITATION OF BRIDGES

Dr. V. Ramakrishnan
Professor of Civil Engineering
South Dakota School of Mines and Technoloy
Rapid City, South Dakota 57701

Summary

A significant portion of the Nation's highway system consists of portland cement concrete pavements. Due to increasing heavy traffic, use of deicing salts, freeze-thaw cycles, studded tires and various other fatigue, surface deterioration and failures are beginning to show up on the pavements. There is a need to determine the most effective and the most economically advantageous means to rehabilitate the distressed concrete pavements.

An investigation sponsored by the Department of Transportation, Washington, D.C., has been completed at the South Dakota School of Mines and Technology, to develop a tough, high-strength, high density, durable concrete for bridge deck construction and a medium strength flowing structural concrete through the use of superplasticizers and steel fibers. The study was made in two phases. The first investigated the basic properties of concrete made with superplasticizers through the use of experimental mixtures conforming to the requirements dictated by statistically valid factorial designs, so that analysis of variance can be used in the evaluation. The second phase extended the findings into an evaluation of superplasticized concrete containing steel fibers. The study has been completed and the significant results are presented in this paper.

In a corrosive atmosphere extensive damage and deterioration takes place in the beams, piers, columns, and abutments of bridges. The most effective way to rehabilitate these bridges is through the use of fiber shotcrete. A suitable mix using new type of steel fibers with deformed ends and glued together into bundles with water soluable adhesive, has been developed for shotcrete work. This paper presents the evaluation of the performance characteristics of this fiber shotcrete.

INTRODUCTION

The causes of concrete deterioration are many and varied. In general concrete not properly designed, prepared, installed, finished or cured is more susceptible to those deleterious influences which cause the concrete to deteriorate and spall and the reinforcing to rust and lose section. This in turn reduces the structural sufficiency and ultimately results in failure. Corrosion of reinforceing steel is the major cause of deterioration in bridge decks, beams, caps, columns, abutments, wing walls and underdecks. Due to increasing heavy traffic, use of deicing salts, feeze-thaw cycles, studded tires and various other fatigue, surface deterioration and failures are beginning to show up on the bridge decks and pavements. Rehabilitation of bridges is one of the most critical problems facing the highway industry. Therefore there is a need to find the most effective and most advantageous methods to rehabilitate the distressed concrete bridge decks and pavements.

The author was given a contract by the Department of Transportation, Washington, D.C., U.S.A., to develop a tough, high-strength, high density, durable concrete for bridge deck construction and a medium strength flowing structural concrete through the use of superplasticizers and steel fibers. The study was conducted in two phases. In the first phase an extensive investigation of superplasticized concretes (both flowing concrete and high strength concrete) was completed, (Ramakrishnan and Coyle 1981) and the second phase extended the findings into evaluation of superplasticized concretes with new type of steel fibers. Highlights of this research are presented in this paper

From 1976, the author and his co-workers have been investigating a new type of steel fibers for mixing with concrete and shotcrete (Ramakrishnan et.al., 1980, 1981). This new type of fiber has successfully eliminated the two major drawbacks of fiber reinforced concrete, namely, "balling" of fibers and lack of adequate anchorage for the wire to develop its potential strength. A suitable mix using this new type of steel fibers has been developed for shotcrete work. This paper presents the performance characteristics of this fiber shotcrete.

SUPERPLASTICIZED CONCRETE

Concrete having good desirable properties in the hardened state is normally made with a low water-cement ratio and with the least possible amount of cement paste in the mix. Such a concrete usually has a low slump and requires intensive and careful compaction like the high-density, low slump concrete used in the Iowa Method of bridge deck resurfacing. In order to produce concrete of the same quality but requiring less vibration, very effective plasticizers, known as superplasticizers have been developed for making flowing and self-compacting concrete. Superplasticizers are added to concrete to cause vast increase in its workability to allow a large reduction in mixing water, and thus produce high-strength concrete. (CAA 1976, Ramakrishnan 1978, 1979, 1980; Malhotra et. al., 1978). Such a change in concrete properties would result in reduced placement costs or reduction in the cement requirement. A well designed mix with superplasticizer will have good flowability and sufficient cohesiveness and would not cause bleeding or segregation or strength reduction either during or after placing of the concrete.

The introduction of superplasticizers has opened up new possibilities for the use of concrete in construction, particularly for bridge deck repair and resurfacing, pavement rehabilitation, and construction of other highway facilities.

In the first phase the basic properties of concrete made with superplasticizers were investigated through the use of experimental mixes conforming to the requirements dictated by statistically valid factorial design so that analysis of variance can be used in the evaluation (Ramakrishnan and Coyle 1981). The statistical design used was the Central Rotatable Composite Factorial Design. The statistical analysis determined the effect of the factors water-cement ratio, cement content, fine aggregate content, and dosage of superplasticizer, and their mutual interactions on the different response variables like workability (slump and flow table spread), plastic unit weight, compressive strength, flexural strength, dynamic modulus of elasticity, and pulse velocity. The homogenity of the variances was ascertained by the standard F-tests. After significant effects due to different factors and their mutual interactions were determined they were used to find suitable response surfaces using curvilinear regression analysis.

The developed prediction equations were used to construct a set of curves for a variety of levels of the four independent variables to readily obtain the estimates of compressive strength, flexural strength, slump, flow table spread and vebe time.

The superplasticizer dosage as a main factor or interaction factor does not have any influence on strength properties of concrete. However, the plastic properties of concrete (slump, flow table spread and vebe time) which are used to measure the workability of the concrete are influenced by the superplasticizer dosage. As a main factor, the fine aggregate content influences air content, plastic and dry unit weight. The compressive and flexural strengths are influenced only by the water-cement ratio and by the interaction of cement content and fine aggregate content. The air content is influenced by three main factors, water-cement ratio, fine aggregate content and superplasticizer dosage. The multiple linear regression model using up to the 4th order terms was found to be an adequate model as indicated by the lack of fit tests.

The analysis of test results has shown that for the same water-cement ratio and superplasticizer dosage, concrete with a lower cement content has higher strength at ages 3, 7 and 28 days. It indicates that for any particular water-cement ratio, there is an optimum cement content which gives best results.

EFFECT OF SUPERPLASTICIZERS ON FRESH CONCRETE

The results reported in this paper are based on the work done by the author using the following two types of superplasticizers: sulfonated naphthalene formaldehyde condensate and sulfonated melamine formaldehyde condensate. The results obtained have established certain real advantages, economical as well as technical that can be gained by the controlled and proper use of these admixtures (Ramakrishnan and Coyle 1981). Compared to the corresponding normal concrete, the concrete with the addition of superplasticizer has good flowability, excellent cohesiveness, less bleeding, no segregation, better pumpability and lower pumping pressure. The entrained air content of fresh superplasticized concrete decreases with time.

Superplasticized concretes exhibit large increases in workability (slump). Since slump loss is an inherent property of concrete, this increase in slump is of short duration, and within 30 to 60 minutes the concrete loses its increased workability. The rate of loss of slump depends on the type of superplasticizer, its dosage rate, temperature of the concrete, the humidity, and the type of cement. Figure 1 shows typical slump loss with time curves for a particular mix with different concrete temperatures. The increase in temperature increases the rate of slump loss. The mixes made at lower temperatures had higher initial air contents and higher slumps whereas mixes made at higher temperatures had very low air contents and low slumps.

While useful trends can be deduced from such visual examination of the slump loss curves, more quantitative parameters are needed. Two such parameters which have been found useful are "slump window" and "total working time." "Slump window" is defined as the time taken for the slump to decay from 76mm to 25.4mm and would be useful to those interested in slipform operations in which high slumps could not be tolerated. The "total working time" is defined as the time needed for the slump to go from the initial value to 25.4mm. These two parameters are plotted in figure 1.

The rapid loss of workability with time is considered a serious drawback. A delay in the discharge of concrete from truck mixer could cause stiffening to the point of unworkability and loss in air content thus affecting the desired air-void system. However this disadvantage can be minimized or mitigated by retempering (adding additional dosages of superplasticizer and air entraining agent). The large increases in workability and the desired air content can be maintained for several hours by the addition of a second and third dosage as shown in Figure 2. The second and third dosages used were less than the initial dosage. The slump after initial mixing was 220 mm whereas, the slump after the first and the second retempering was 191mm. The rate of slump loss was low and the time taken to reach a workable slump of 51mm after each retempering was 3.5 hours. Overdosing of superplasticizer should be avoided because this can cause segregation.

EFFECT OF SUPERPLASTICIZERS ON THE PROPERTIES OF HARDENED CONCRETE

With the addition of superplasticizers, water reductions of up to 20 percent can be achieved in the manufacture of concrete. This causes an increase in mechanical properties like compressive strength, flexural strength, and modulus of elasticity. This increase in strength is generally proportional to the reductions in water to cement ratio. The ability of superplasticizers to reduce water and achieve very high strengths is of special importance for the concrete repair and rehabilitation work where high early strengths are needed.

In some cases, the concretes with superplasticizers had shown higher strengths at early ages than the reference concretes indicating an increased rate of strength development at early ages.

The study has shown that the shrinkage of superplasticized concrete is equal or less than the shrinkage of reference concrete. The concrete with the superplasticizers has approximately the same creep as the reference concrete. The study has shown that the addition of superplasticizer does not affect the relationship between the accelerated and 28-day compressive strengths (Ramakrishnan et. al. 1979). The modified boiling test, ASTM C684 was the accelerated strength test used in the study.

In cold regions, the resistance of concrete to freeze-thaw cycling is important. Therefore for concretes used in the cold regions, an air entraining admixture is added to entrain air bubbles of the required sizes. These air bubbles provided the satisfactory freeze-thaw durability of the concretes. An extensive investigation has shown that the concretes with superplasticizers have adequate freeze-thaw durability in spite of the larger bubble sizes found in

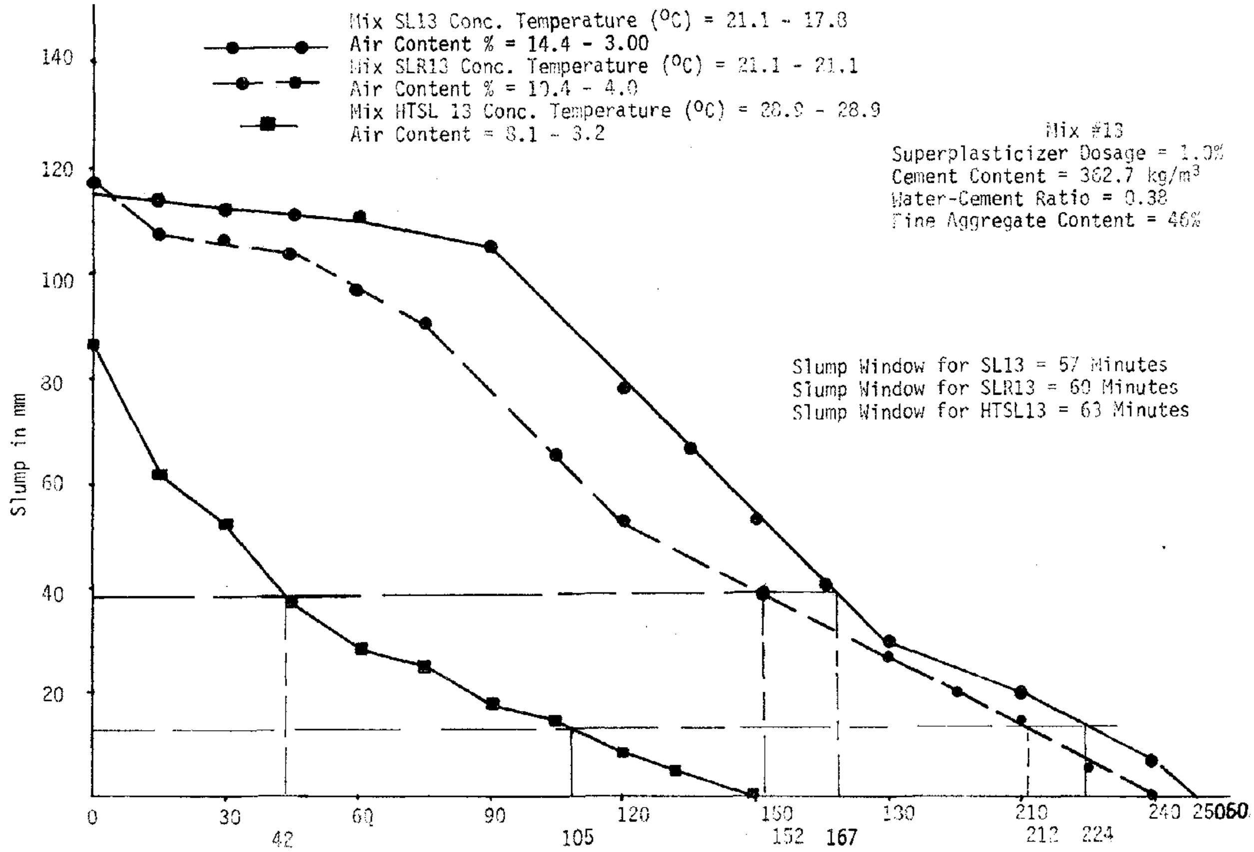


Fig. 1 Slump Versus Time

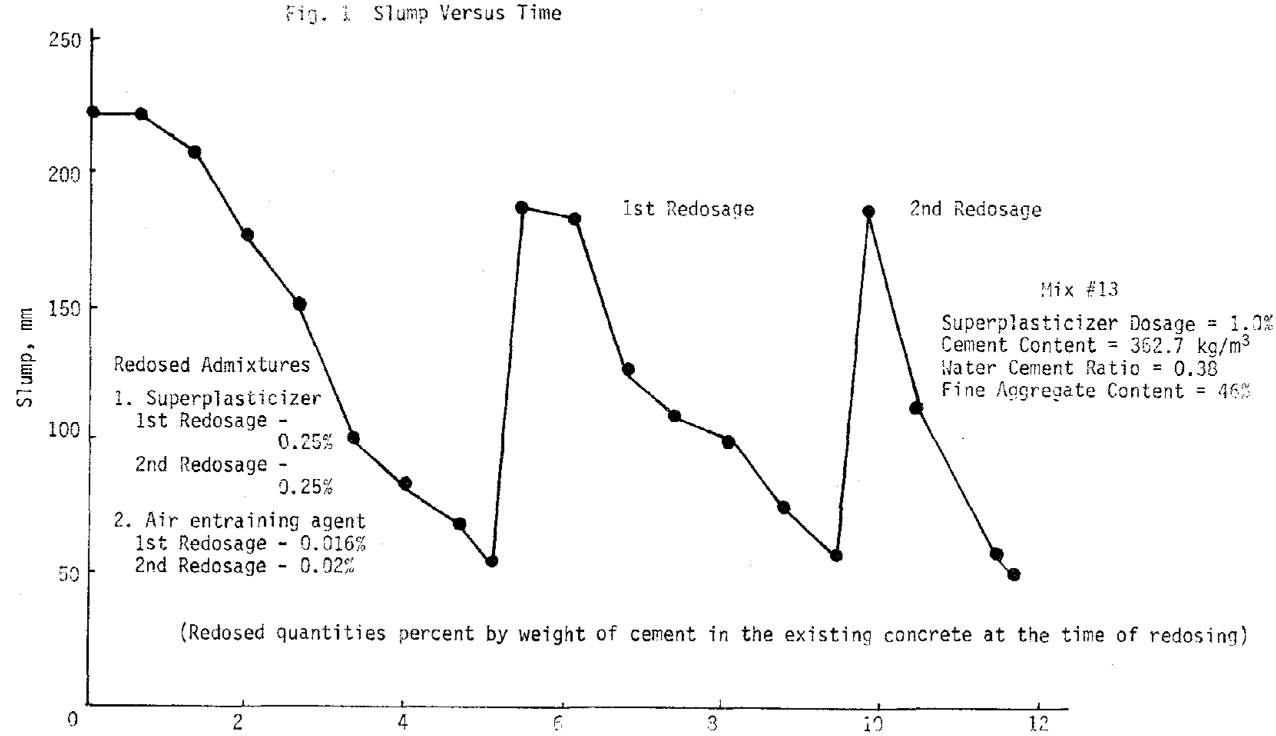


Fig. 2 Slump-Time-Retempering Study for Mix RTP13

some concretes. The superplasticized concretes had performed equally well in the deicer sealing test recommended by ASTM.

In general the study has shown that with the addition of superplasticizers, it is possible to produce a very highly workable concrete, which is known as flowing concrete, without any detrimental effects either in the plastic or in the hardened states. It is also possible to produce extra high strength concretes (80 to 100 MPa) with low water cement ratio of 0.25 to 0.28 and high workability.

SUPERPLASTICIZED FIBER REINFORCED CONCRETE

High strength concrete is being increasingly used in the repair and rehabilitation of bridges (particularly overlays), buildings, and other reinforced and prestressed concrete structures. One major drawback of high strength concrete is that it is brittle and leads to a sudden and explosive type of failure. The failure will be catastrophic, particularly in structures which are subjected to earthquakes, blast or suddenly applied loads. An ideal solution to overcome this serious disadvatnage of high strength concrete is to add steel fibers in the concrete. It is well established (Ramakrishnan et. al. 1980,1981) that the addition of steel fibers greatly increases the ductility, the energy absorption capacity, and the ultimate strain capacity of the concrete. Fiber reinforcement considerably increases the ultimate flexural strength, the post-crack load carrying capacity, impact resistance, shear and torsional strength, fatigue strength, shock resistance, and failure toughness. However, the main problems associated with fiber concrete are fiber balling and inadequate workability. Balling of fibers in the mixer prevents uniform distribution and also causes problems when the concrete is placed. Fiber balling and consequent inadequate mix workability imposes an upper limit beyond which increase in strength and other properties are no longer realized when using conventional mixing procedures.

In this study, in order to remedy the problem of fiber balling, new type of fibers (several fibers with hooked ends glued together side by side with water soluble adhesive) were used with successful results, and a superplasticizer was used to increase workability adequately. The addition of fibers and superplasticizers proved to be an ideal combination to produce high strength ductile concrete.

The effects of various parameters such as water-cement ratio, cement content, fiber content, superplasticizer dosage and air-entraining agent on the workability and strength of fiber reinforced superplasticized concrete were determined.

The hooked fibers used in this project are glued together side by side into bundles with a water-soluble adhesive. These fibers are made from low carbon steel wires, and have a nominal length of 52 mm and a diameter of 0.5 mm. The bundling of fibers creates an artificial aspect ratio (the ratio of length to diameter of the wires) to approximately 30 when introduced to the mix. When the glue is dissolved by the water in the mix, the fibers will be separated as individual fibers with an aspect ratio of 100. These hooked fibers are commercially known as "Dramix."

For the same water-cement ratio, an increase in superplasticizer dosage caused considerable increase in slump and air content. With higher cement content (390 kg/m^3) and higher superplasticizer dosage (1.2%) it was possible to get up to 200 mm of slump at low water-cement ratio (0.32) without any segregation. The measured slumps for superplasticized fiber reinforced concrete was slightly less when compared to the values of superplasticized concrete without fibers.

The hooked fibers performed well during mixing because no balling occurred in almost all the mixes except for mixes with zero slumps and high fiber contents, even though the fibers were charged to the mixer all at one time along with the aggregates. This must be taken as the consequence of low aspect ratio created by the collating of the fibers. It took approximately 1½ minutes for the glue to dissolve and for the fibers to separate. Of the workability tests commonly used, the slump test is the most sensitive indicator of changing workability since the range covered is so large. However, the flow table test represents a little closer the real placing situation and, hence, a more realistic indicator of workability for flowing concrete than the slump test. The relationship between slump and flow table spread for plain and fiber reinforced superplasticized concrete is shown in Figure 3. These two curves show only a small difference between them; the relationship between slump and flow table spread is approximately the same for both superplasticized concretes, with and without fibers.

Concrete with superplasticizer added and having a slump of 200 mm (7.9 in.) or greater and a flow table spread within the range 510 to 620 mm is classified as a flowing concrete (CAA, 1976). Flowing concrete should not exhibit excess bleeding or segregation. Other terms used to describe the flowing concrete are: self-compacting concrete; flocrete; soupcrete, liquid, fluid and collapsed-slump concrete. This study has shown that it is possible to produce flowing fiber reinforced concrete.

Unlike for plain superplasticized concretes, the measured vebe times were quite high for superplasticized fiber reinforced concretes. For superplasticized fiber reinforced concrete mixes with zero slumps, they were approximately in the range of 10 to 28 seconds, whereas for superplasticized concretes without fibers they were in the range of 7 to 10 seconds.

The increase in fiber content from 32.63 kg/m^3 to 56.36 kg/m^3 had negligible effect on the slump and flow table spread. However, an increase in fiber content caused an increase in vebe time. Slump and flow table tests are used to measure the workability of concrete based on the flowability character of concrete. Hence, an increase or decrease in fiber content did not have much influence on the results attained from these tests. Vebe time is used to measure the workability of concrete, based on the energy needed to compact the concrete. The energy requirement for plain superplasticized concrete mixes is less than for superplasticized fiber reinforced concrete. In the case of superplasticized fiber reinforced concrete mixes, the energy needed seems to be proportional to the fiber content in the concrete.

Hardened Concrete Properties

Compressive Strength: The analysis has shown that for the same water-cement ratio and superplasticizer dosage, concretes with a higher cement content showed higher strength, and an increase in superplasticizer dosage caused an increase in compressive strength.

An increase in fiber content did not cause any appreciable change in compressive strength.

Flexural Strength: When all other factors were the same, an increase in fiber content caused an increase in ultimate flexural strength. However, the increase in flexural strength was only about 10 percent for an increase in fiber content from 36.63 kg/m^3 to 56.36 kg/m^3. The flexural strength values varied in the same way as compressive strength values, with respect to variation in all factors, except in the case of fiber content. For a given compressive strength, the corresponding flexural strength was greater for superplasticized fiber concrete

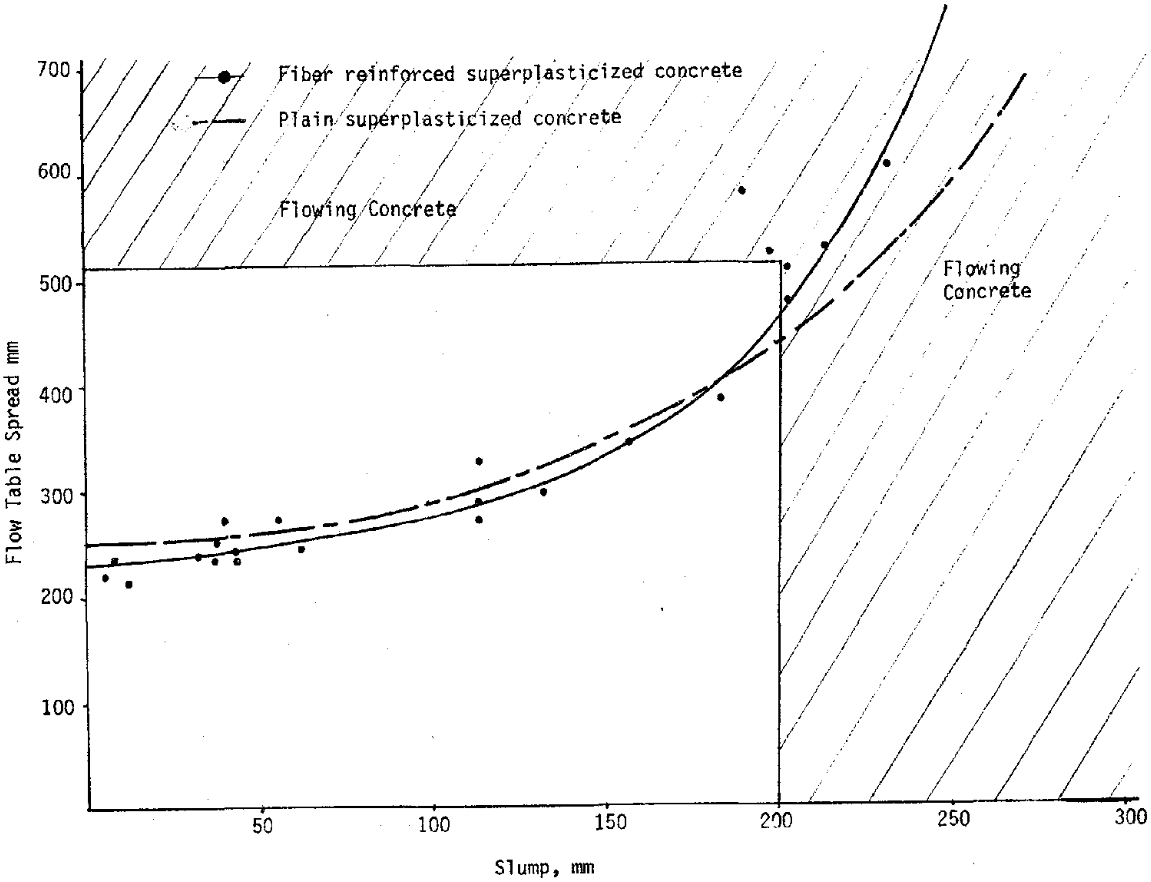


Fig. 3 Relationship Between Slump and Flow Table Spread

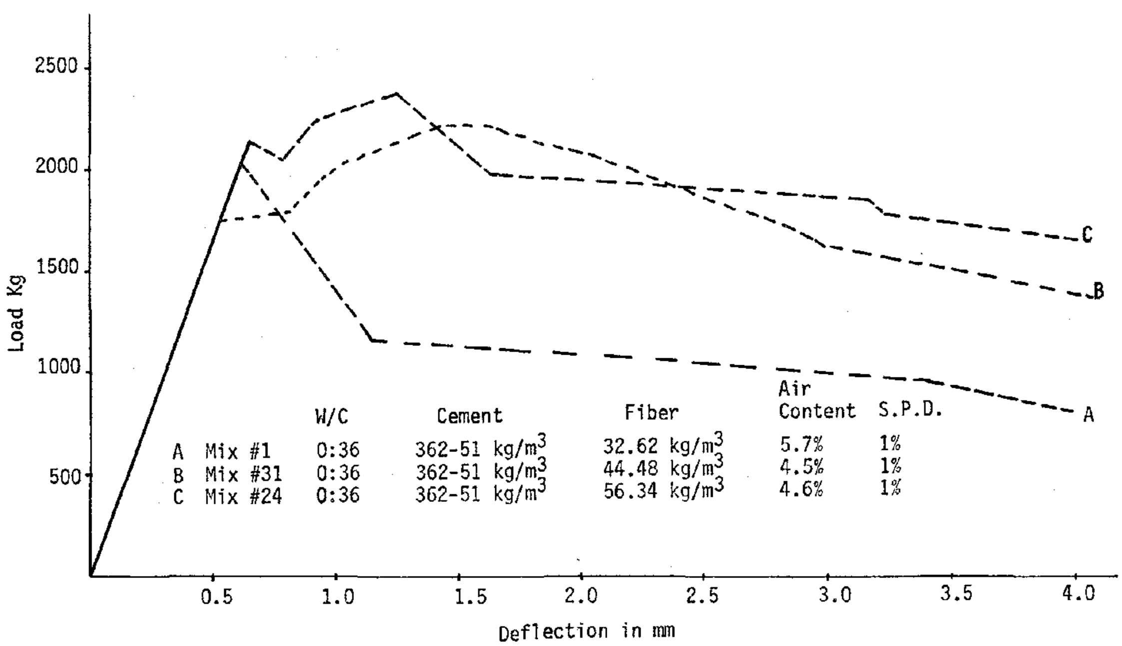


Fig. 4 Comparison of Load Deflection Curves for Mixes with Different Fiber Content (7-Day)

when compared to plain superplasticized concrete.

A significant difference in the performance of plain superplasticized concrete and superplasticized fiber reinforced concrete was found in the static flexural test. The hooked fibers had proved their ability as crack arrestors. The cracks were prevented from propagating until the composite ultimate stress was reached. The mode of failure was simultaneous yielding of the fibers and the matrix. During the test one could actually hear the popping sound of the fibers failing in tension and not due to the bond failure. It seems obvious that the deformed ends contributed significantly to the increase in bond between fiber and matrix. The significance of good bond can be seen from the typical load deflection curves (fig. 4). The curves indicate a ductile behavior. These curves also show the advantage of fiber superplasticized concrete versus non-fiber superplasticized concrete in obtaining higher flexural strength, higher toughness, and high energy absorption qualities.

Toughness Index: The toughness index is a measure of the amount of energy required to deflect the 102x102 mm beam, a given amount, compared to the energy required to bring the beam to the point of the first crack. It is calculated as the area under the load-deflection curve up to 1.9 mm, divided by the area under the load-deflection curve of the fibrous beam up to the first crack strength (proportional limit, defined as first deviation from linear).

In general, the toughness index for the fiber reinforced superplasticized concrete varied greatly depending upon the position of the crack and the distribution of fibers. An increase in fiber content caused an increase in toughness index; the toughness index increased from 4.75 to 6.5 for an increase in fiber content from 32.63 to 56.36 kg/m^3.

All specimens made of plain superplasticized concrete failed immediately after the first crack and hence the toughness index for these specimens is equal to 1.

The fiber reinforced concrete shows a tremendous ability to absorb impact loading. Concrete with fibers had 12 times higher impact strength. The fiber reinforced superplasticized concrete has less shrinkage and comparable creep as that of the corresponding superplasticized concrete without fibers. The fiber reinforced superplasticized concrete has excellent deicer sealing resistance. An important advantage is that the freeze-thaw resistance is increased with the addition of steel fibers.

FIBER SHOTCRETE

Shotcrete is excellent for restoration and repair of concrete, repairing fire damage and deterioration, and waterproofing of walls. It provides long-term steel corrosion protection of piling, coal bunkers, oil tanks, smokestacks, steel building frames, and other structures, as well as encasing structural steel for fireproofing. (Crom 1981) Shotcrete repair is extremely effective for variable depth repairs of bridge beams, caps, columns, abutments, wingwalls, and underdeck, whose deterioration has been accelerated by the introduction of continuous beam designs, shallow depth deck slab, asphalt surfacing and the heavy use of deicer salts (Glassgold, 1980).

The addition of steel fibers to shotcrete makes it almost an ideal material for concrete repair and renovation. It gives shotcrete new characteristics unknown so far; steel fibers make shotcrete a tenacious material. Steel fiber shotcrete maintains its load carrying capacity even after the first fissures and cracks have occurred due to increased load. Its capability of absorbing strain energy under continuing deformation is up to 20 times greater than that of plain shotcrete. Steel fiber shotcrete resists far greater impact and shock and it is highly resistant to abrasion and it is more weatherproof than plain shotcrete. There is a considerable reduction in drying shrinkage which is important in repair and rehabilitation work. Steel fiber shotcrete has better heat resisting capacity and refractory linings containing fibers live up to four times longer.

Steel fiber shotcrete saves labor and materials because faster operation is possible as installation of mesh is avoided and thinner sections can be used. Steel fiber shotcrete makes the operation safer because the interlocking steel fibers give the shotcrete an instant apparent strength, earlier support and higher ultimate load carrying capacity.

Steel fiber shotcrete with its excellent ductility is better suited for repair and reconstruction of bridges and buildings damaged by earthquakes. It has an advantage in applications where there may be relatively large deformations like tunnel and mine linings. Because of its increased resistance to blasts and shocks, and the preservation of structural integrity despite extensive cracking, fiber shotcrete is highly recommended for military structures.

Steel fiber shotcrete was first placed in the United States in 1971 and it has been used in Germany, Sweden, England, Norway, Finland, Switzerland, Poland, South Africa, Australia, Canada, and Japan (Henager, 1981). Existing equipment with little or no modifications can be used for the application of steel fiber shotcrete.

Research Program

In an investigation conducted at the South Dakota School of Mines and Technology, (Ramakrishnan et. al. 1981) the performance characteristics of fiber shotcretes with four types of steel fibers were evaluated and they were compared with those of conventional concrete. The influence of the quantity of fibers on the properties of hardened shotcrete was also determined.

Materials and Mixes

The fibers used were two types of collated steel fibers with deformed ends (Dramix ZP 30/50 and Dramix 30/40), straight steel fibers (Fibercon), and cut steel tire cord. The Dramix fibers were glued together in bundles creating an artificial low aspect ratio (20 for ZP 30/50 amd 25 for ZP 30/40) when introduced to the mixer. When the glue is dissolved, the bundled fibers will separate into individual fibers with a high aspect ratio (60 for ZP 30/50 and 75 for ZP 30/40).

Type III (High-Early Strength) Portland Cement was used for all the mixes. The fine aggregate used was natural sand with a water absorption coefficient of 1.6, and a saturated surface-dry specific gravity of 2.62. The basic mix proportions used are given below:

	Fibers (kg/m^3)	Cement (kg/m^3)	Fine aggregate (kg/m^3)
No Fiber	-	442	1618
0.6%	46.5	442	1599
1.0%	78.5	442	1589
1.3%	102.5	442	1579

Notes: 1. All Weights are SSD
2. Assumed water/cement ratio = 0.46
3. Assumed entrapped air = 5%
4. Fine aggregate absorption = 1.6%
Bulk Sp. gravity = 2.62

A total of 11 different shotcrete mixes were investigated.

The batching procedure consisted of placing the fine aggregate and fibers into the concrete mixer and premixing for about 2 min. The premixing was used to ensure even distribution and prevent balling of the straight fibers in the shotcrete mixer. The premixing separated an estimated 40 to 50 percent of the collated fibers. The cement was then added to the mixer and ASTM Standard mixing procedure of 3 min. mixing, 3 min. rest, and 2 min. mixing was used. The capacity of the concrete mixer used was 0.17 m^3; however, 0.096 m^3 was mixed at a time so that one full sack of cement could be used per batch. After the premixing, the material was transported by wheel barrow and fed into the "Reed Lova II" shotcrete machine, which had a 20 pocket feed wheel, 51 mm material hose and 51 mm nozzle. The machine was operated by a 17 m^3 capacity air compressor which supplied approximately 0.69 MPa pressure to the shotcrete machine.

The shotcrete mixes were pneumatically sprayed into the test panels using the "dry-mix process", in which water was added at the spray nozzle in a number of fine streams. As the material passed through the final 105 or 126 mm of the nozzle, it was mixed with the water.

The shotcrete mixes were sprayed into the test panels by a well known shotcreting company using a competent nozzleman, who has 18 years of experience in shotcreting.

Observations and Test Results

Rebound is defined as the coarse particle, and fiber that ricochet (rebound) from the surface. The amount of rebound measured was approximately 5 to 15 percent, which is well within the approximate values of 15 to 30 percent measured for conventional shotcrete. For the glued fibers, all fibers separated by the time they hit the panel. Looking at the surface, an excellent distribution of fiber and aggregate was noted. Inspection of the cut specimens showed some laminations where the panels were built up to less than full depth initially and additional shotcrete was applied a few minutes later to obtain the required depth.

In order to determine the fiber distribution in the hardened shotcrete, specimens were cored from panels and x-ray photographs were taken. These photos showed that there was a good and uniform distribution of fibers in all directions.

This investigation had clearly shown that collated and hooked steel fibers can be successfully used in field applications with a conventional shotcrete machine. The premixing helped to distribute the fibers and to prevent balling of the fibers in the feed wheel of the shotcrete machine.

The beneficial characteristics of fiber shotcrete were clearly demonstrated in the flexure test. Non-fiber shotcretes were brittle and failed suddenly with a toughness index of 1 and no additional loadcarrying capacity or energy absorption. Straight fibers with 0.6 percent concentration behaved similar to the nonfiber shotcretes; however, straight fibers with 1.0 percent concentration had slightly improved toughness and energy absorption characteristics. Shotcretes with hooked fibers had substantially higher load-carrying capacity after the development of substantial cracks and deformation. These shotcretes had a very high toughness index indicating excellent energy absorbing capacity.

The flexure test has shown that an excellent end anchorage is established between the hooked fibers and the matrix, resulting in a high ductility of the composite material. A good bond was also established between the brass plated cut steel tire cord and the matrix. The addition of fibers had increased the ultimate flexural strength of the shotcretes. The increase was modest with increased quantities of steel fibers. Shotcretes with hooked fibers had the highest increases in flexural strengths..

Fiber shotcretes showed an ability to control cracking and deformation under impact loading. Shotcretes with hooked fibers showed a tremendous ability to absorb impact loading. For higher fiber contents, the impact resistance was dramatically increased. Though not as high as the hooked fiber shotcretes, the cut steel tire cord and straight fiber shotcretes also showed an ability to absorb higher number of blows than the nonfiber shotcretes. When the shrinkage values were compared on the basis of a standard compressive strength then it was seen that the addition of fibers reduced the drying shrinkage values; the reduction is proportional to the quantity of fibers added.

A comparison of 38 mm and 76 mm thick shotcretes had shown that the flexural strength of thinner shotcretes were considerably higher, whereas the compressive strength and impact resistance were lower than the 76 mm thick shotcretes.

CONCLUSIONS

Substantial improvement in workability with consequent reduction in placement costs, or a considerable saving in cement content are the realistically achievable advantages through the use of superplasticizers for whatever purpose conventional concrete is being used. Superplasticized concretes with low water cement ratios and high early strengths are particularly suitable for concrete repair, and rehabilitation of bridges and other structures.

The addition of the special type of steel fibers (with deformed ends and glued together into bundles with a quick water soluable adhesive) to superplasticized concrete greatly increases its ductility, toughness, impact resistance, ultimate flexural strength, post-crack load carrying capacity, shear and torsional strength, fatigue strength and shock resistance. This is achieved without a reduction of workability or the usual "balling" of steel fibers. Therefore the fiber reinforced superplasticized concrete is an almost ideal material for the rehabilitation of concrete structures.

Whenever shotcrete is used in construction for repair or rehabilitation work, it is recommended that steel fiber shotcrete should be used because of the improved performance of fiber shotcrete. With the addition of steel fibers it is possible to reduce the thickness of shotcrete substantially.

ACKNOWLEDGMENTS

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Josifek of the Bekaert Steel Wire Corporation, Chicago, U.S.A. The opinions expressed in this report are those of the author and not necessarily those of the U.S. Department of Transportation.

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TITLE: REHAB OF VINTAGE WAREHOUSE INTO A CUSTOM COMMERCIAL FACILITY

AUTHORS: KISHOR S. MEHTA
PRINCIPAL, RESTL DESIGNERS
CONSULTING ENGINEERS
GAITHERSBURG, MARYLAND 20877

JOSEPH WISNEWSKI
PARTNER, LEWIS/WISNEWSKI & ASSOCIATES, LTD.
ARCHITECTS, ENGINEERS & PLANNERS
ALEXANDRIA, VIRGINIA 22314

SUMMARY

This is a historic building over 100 years old, located in the City of Alexandria, primarily used as a warehouse in the past and presently not in use. Existing building after full investigation, now converted into a comfortable office building for daily operation of National Association of Chain Drug Stores. In present economical condition renovation of such older buildings into new commercial or residential property is getting widely popular among developers. Real estate value of this building after renovation had doubled in three (3) years. This paper presents 1) History of location, 2) history of building, 3) new design, and 4) conclusion.

PROJECT STATEMENT

A Trade Association purchased this warehouse building from a developer for renovating into an office building. The building is a detached part of a larger residential scale office development located in an area of neo colonial townhouses.

Although the building is listed as being over 100 years old, a major fire and subsequent changes obligerated any significant architectural features.

After the space requirements were decided, instructions from the owner were to provide a pleasant working environment that encouraged the professional communication among the staff. This was to be accomplished without sacrificing privacy.

The existing building had a large open space with four walls. All exterior walls were of battered brick construction with a maximum thickness of 3'-0" with minimum penetrations.

In the new design, exterior openings were minimized, with vistas focused inward to an atrium and an open stair. A serious attempt was made to provide exterior wall openings in keeping with the building volume and compatible, without copying, to the existing neighborhood.

HISTORICAL BACKGROUND OF PROJECT SITE

City of Alexandria, Virginia, is located across the Potomac River from the Capital of the United States, Washington, DC. It is only six miles from the White House, a U.S. President's residence.

Alexandria was never destined to be a great commercial and industrial center. It was not created by successive explosive growths, characteristic of so many American cities, which destroyed the evidences of earlier building. Instead, Alexandria formed more gradually as a collection of residences of architectural quality with sufficient commercial entities to serve those residences.

The Congressional action of March 1791, which included Alexandria in the new District of Columbia, prohibited the construction of public buildings on the Virginia side of the river and thus strengthened the residential character of Alexandria. The announced intention of sitting the new capital across the river caused the first noticeable building boom in the last decade of the 18th century. Curiously enough, the second such surge came within a few years after the retrocession of the Virginia portion of the District of Columbia to that state in 1846. Just prior to the Civil War, within a period of some forty-eight months, the city added over seven hundred buildings to its inventory.

Conservative taste in Alexandria carried 18th century stylistic details into the 19th century. The tendency, therefore, had been to think of the older part of the city as being solely 18th century. This concept, of course, obscures the rich continuum of daily life over many decades of the 18th and 19th centuries which the buildings actually reflect. This misconception was heavily reinforced by the presence of George Washington as a prominent 18th century figure in the town and by the continued existence of a number of landmark buildings such as Gadsby's Tavern and Christ Church with which he is irrevocably intertwined.

As a result, such landmark buildings as these, and those associated with Robert E. Lee and other prominent Alexandrians, serve as pivotal points in an area that has a highly developed sense of identity and place. However, without the many supporting buildings of age which line the streets of the city, such landmarks would stand as isolated and forlorn entities, unrelated to their environment.

As Alexandria enters its third century, it retains to a remarkable extent the continuity of its early streetscapes. It has largely survived the onslaughts of the throw-away mentality of our 20th century society. If anything, there appears to be a growing sensitivity to the overall values that intensify the quality of "place" in the city. Restoration and rehabilitation continue. But there is also a new awareness of the importance of the amenities of the urban streetscape. The city is planting trees, laying brick sidewalks, burying utility lines, and increasing its control over advertising graphics.

What one sees when walking the streets of Alexandria, unfolding from the river to the railroad, is a reflection of time and growth. Collectively, these individual units range from local to national in historical significance and from vernacular to sophisticated in architectural design. As a total conservation neighborhood, the Historic District has few peers in the country, and certainly none in the greater Washington area. The local government sensed this in 1946 when it passed the local zoning ordinance defining the Old and Historic Alexandria District, restricting the changes permitted by private property owners to what was then considered to be an inordinate degree. That this was one of the first such local actions in our country is noteworthy.

Higher government authorities subsequently confirmed the importance of preserving this neighborhood. The Department of the Interior in 1966 applied National Landmark status to the port area. In 1969, upon application by the Commonwealth of Virginia, the Department of the Interior included the entire district on the National Register of Historic Places.

The project is located near the intersection of North-Lee and Oronoco streets, Alexandria, Virginia, and close to the landmark buildings - Gadsby's Tavern and Christ Church.

HISTORY OF THE BUILDING

The original structure was constructed approximately in 1860 as part of the original Alexandria Gas & Electric Works. Through the years it changed hands but the use remained essentially the same, a warehouse.

In 1976 the property was sold to Design Resources Incorporated, with the intention of developing into a commercial or residential complex.

The architectural firm of Lewis/Wisnewski and Associates, Ltd. was commissioned to prepare feasibility studies on alternative solutions. It was determined that the land was too valuable for development into a residential project. Commercial use appeared to be the only feasible possibility.

The architect prepared several alternate schemes for the facility. The developer contacted various trade and professional associations in the Metropolitan Area for the possibilities of purchasing the new facility after it is completed.

In 1977 the National Association of Chain Drug Stores negotiated a deal with the developer to develop the property as their National Headquarters.

The existing warehouse building is about 50' x 66' and approximately 32 feet in height to the ridge of the roof with a clear ceiling height of 22 feet.

The existing exterior walls' thickness varied from 3 feet at the ground level to about 12 inches at the roof. The walls were built out of solid fire bricks and were in generally good condition except that at few locations some pointing of the existing walls was required. The existing roof was steel trusses spanning 50 feet and spaced approximately at 15 feet apart. The existing trusses were in good condition and need not require repair or replacement.

The existing building was not occupied for a long time. The wall foundation appeared to be crack free. The width of the wall footings was assumed and later verified as same thickness as wall at ground floor. But the soil capacity was very low and a maximum load capacity of the footing was determined on the basis of the available soil value.

Field survey of the existing structure and ascertaining its soundness is extremely important in a renovation project to estimate the cost and the time required to complete the project. You may run into "surprises" at times in some complicated projects in spite of careful field investigation. A higher contingency in the budget is usually allocated for any renovation project to account for "surprises" than a new project.

The existing building had a minimum unpaved parking facility available.

Substantial changes in the design of the existing building were required to meet the proposed design needs in accordance with the local building and zoning ordinance for preservation of historic buildings.

RENOVATED BUILDING

The building shell covered approximately 3,300 gross square feet and was 22'-0" high from truss bearing to top of existing floor slab. This clear height allowed the construction of 2 floors and a basement with windows above grade that could be used for support facilities or future offices.

The new square footage of the building was limited by the amount of parking that was provided and the floor to land area ratio that was established by the governing agencies due to the zoning restriction. Due to these factors the basement had a 7'-0" ceiling height which allowed the installation of the basement but did not violate the parking requirements or the floor area ratio as the city code permits a ceiling height of less than 7'0" as non-office use space.

Because the building was designed around a central atrium and had an open stair, the code required that a sprinkler system and a smoke and heat detection system be installed. This plus a second fire stair allowed the open plan design concept to be used and made it feasible to convert the building to a modern office use.

The building maintains a low profile as it is nested withing a residential area. Small windows of 1" solar bronze insulating glass and anodized aluminum casement frames dot the perimeter. The only major commercial element is the main entrance with its vertical and recessed, full height glass wall.

The building was serviced by one elevator, an open stairwell serving the first and second floors and a second enclosed stairwell servicing the three floors. A massive oak handrail was the prominant feature on the segmented stair.

It was proposed to construct two additional framed floors between existing ground floor and the bottom of the existing truss spanning between existing wall and grade beams at the center of the building. New columns were supporting the grade beams. New footings were provided at the columns. The footings were designed for a soil value of 2,000 pounds per square foot. A 7" reinforced framed slab was provided at the ground floor. A careful analysis was required to locate the additional interior columns so that the existing structure was not overstressed. With this criteria in mind, the new columns were located which also had to meet the architectural design.

The existing wall footings were at elev. 9.00. The new floor slab was provided at elev. 13.85. New slab at ground floor was keyed into existing wall by removing existing brick to provide a minimum of 2 inches of seat.

Three different framing schemes were investigated for comparison.

<u>Scheme 1: A one way concrete slab and beams with an interior row of columns:</u> The columns at the interior row were spaced approximately at 15 feet to 20 feet on centers. The concrete beams were spanning between existing exterior walls with center support at columns. The beams on walls were supported by removing part of the wall to provide bearing for the beams. The slabs were spanning between beams having 15 feet to 20 feet spans.

<u>Scheme 2: Composite Steel Constuction</u>

Floor construction consisted of 3 3/4" inches thick light weight concrete slab reinforced with welded wire fabric over 2 inches deep steel composite deck supported on composite steel beams spaced at 7'6" on centers. The deck was welded to steel beams. The composite steel beams were supported on girders between steel columns. Steel studs were encased in the slab and welded to beams to achieve composite action.

<u>Scheme 3: Open Web Steel Joists Supported on Existing Walls and New Steel Beams:</u>

Floor construction consisted of 2 1/2 inches normal weight concrete slab reinforced with welded wire fabric over 5/8 inches deep corrugated steel form supported on steel joists spaced at 2 feet on centers. The steel form was spot welded to joists. The joists were supported on steel beams and on existing walls. The beams were supported on columns. A 10 inches deep continuous steel channel was anchored to existing wall with 3/4" diameter expansion bolts at 2 feet on centers to support the steel joists.

The materials selected for the above schemes were as follows:

Concrete: fc'=3000 psi (fc' - a 28 day compressive strength).
Structural Steel: ASTM A-36 or ASTM A-572
Reinforcing Steel: ASTM A-615 grade 60.
Welded Wire Fabric: ASTM: A-185.

First and second schemes were not only expensive but were overstressing the existing wall foundation due to heavier dead loads. The third scheme was selected for the project for its simplicity and less dead weight. It was also the most economical scheme. A considerable erection time was saved by adopting this scheme.

The basement was used for general storage and first and second floors are used for offices. Mechanical equipment room was on the basement floor. The mechanical equipment selected were electric since gas was not available in this area. These electric units were fully decentralized except for the condensing water.

Floor mounted water to air heat pump units were used for individual offices on first and second floor. These units were actually complete heating/cooling units providing individualized temperature control. These units take outside air through wall to meet the minimum ventilation requirements in each room. These individual units eliminate the need for centralized air conditioning unit, thus avoiding large ducts and lower ceiling heights.

Floor mounted heat pumps (located in Mechanical Equipment Room on Ground Floor) were used to serve general storage area on ground floor and conference room on first floor. These pumps were individually controlled to maintain better efficiency and economy.

Three separate ceiling hung units (one on first floor and two units on second floor) provide heating/cooling for other interior areas, because of the limitation of ceiling space.

Closed circuit type cooling tower was provided to cool approximately 86 GPM of water from 103.8°F to 91.5°F at 78°F WB ambient temperature. This is the source of water used for all heat pumps installed in the building.

A packaged electric hot water boiler with a capacity to heat 86 GPM of water from GA 2°F to 70°F was provided to meet hot water requirements.

Conclusion

Architecturally the facility is a comfortable setting for the daily operations of the National Association of Chain Drug Stores. The building represents a low-key but understated elegant surrounding for the occupants.

In present economic situations where mortgage money is extremely expensive to obtain, renovation makes sense. The recent trend is "buy not build." It has advantages such as good solid buildings in less time and at less cost. Another "fringe benefit" out of such renovation of older buildings is the preservation of a "Historic Monument" which is a good incentive for the developer from the investment point of view because of tax benefits legislated by the U.S. Congress. Most of the older buildings have larger safety factors in their design and perform quite effectively in adverse conditions like wind storms and earthquakes.

An old unoccupied warehouse building was turned into a three-story modern office building.

A centralized heating/cooling system was considered but discounted because of space conditions; thus, eliminating the need for larger mechanical eqipment room and lower ceiling heights. Also, seasonal efficiency of central heating/cooling system for this kind of office building may range from 40% to 60% whereas the efficiency is almost 95% for the decentralized units. This will result in eliminating standby and partial load losses.

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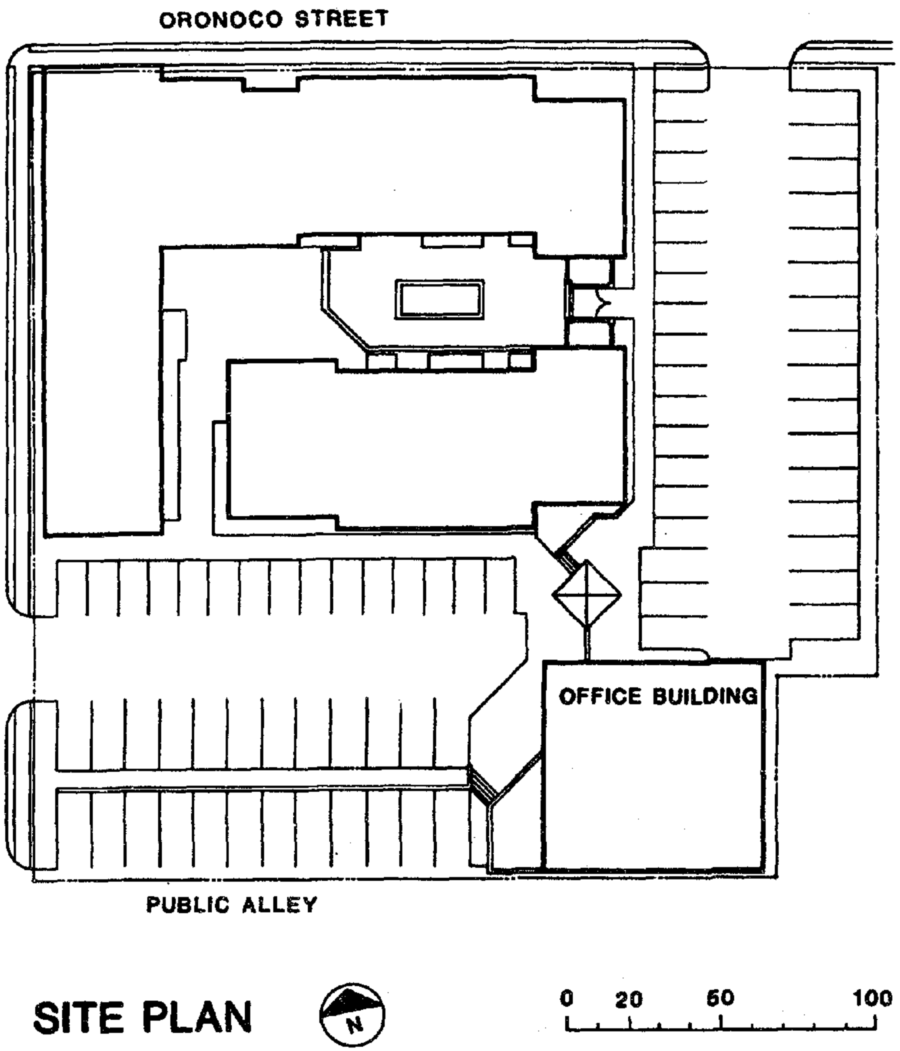
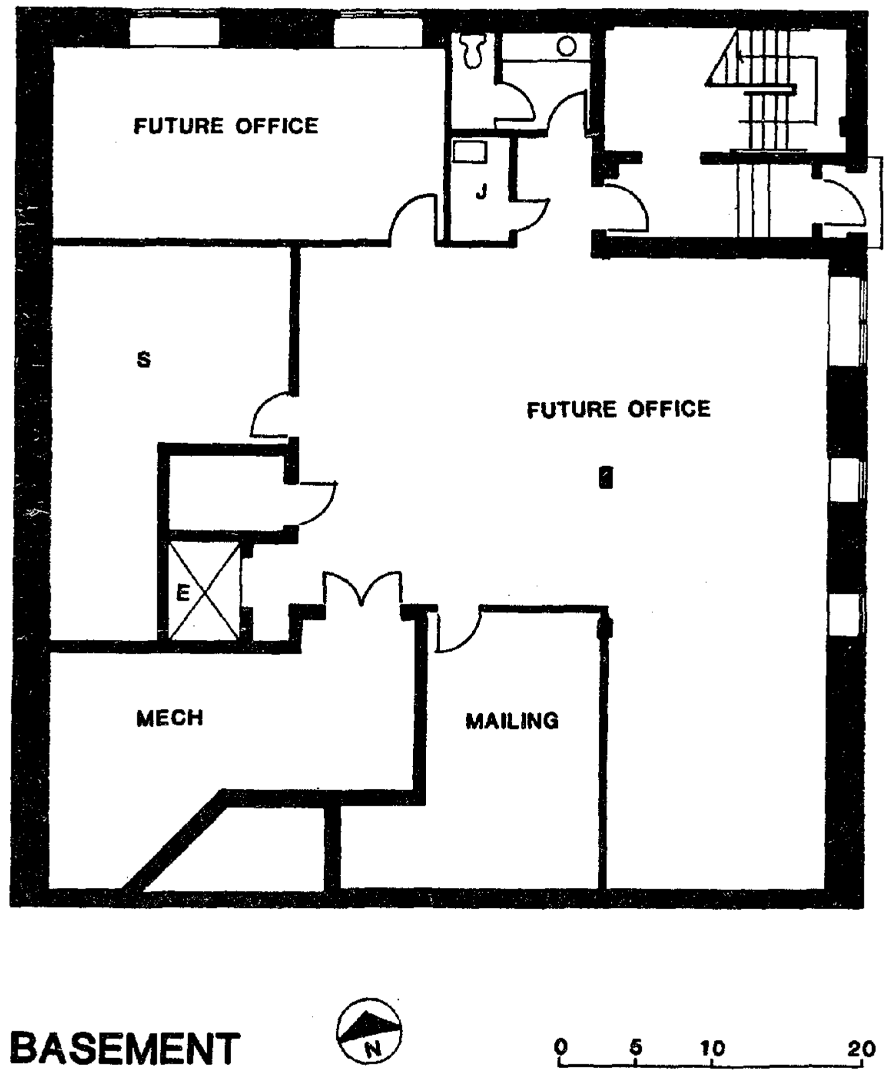
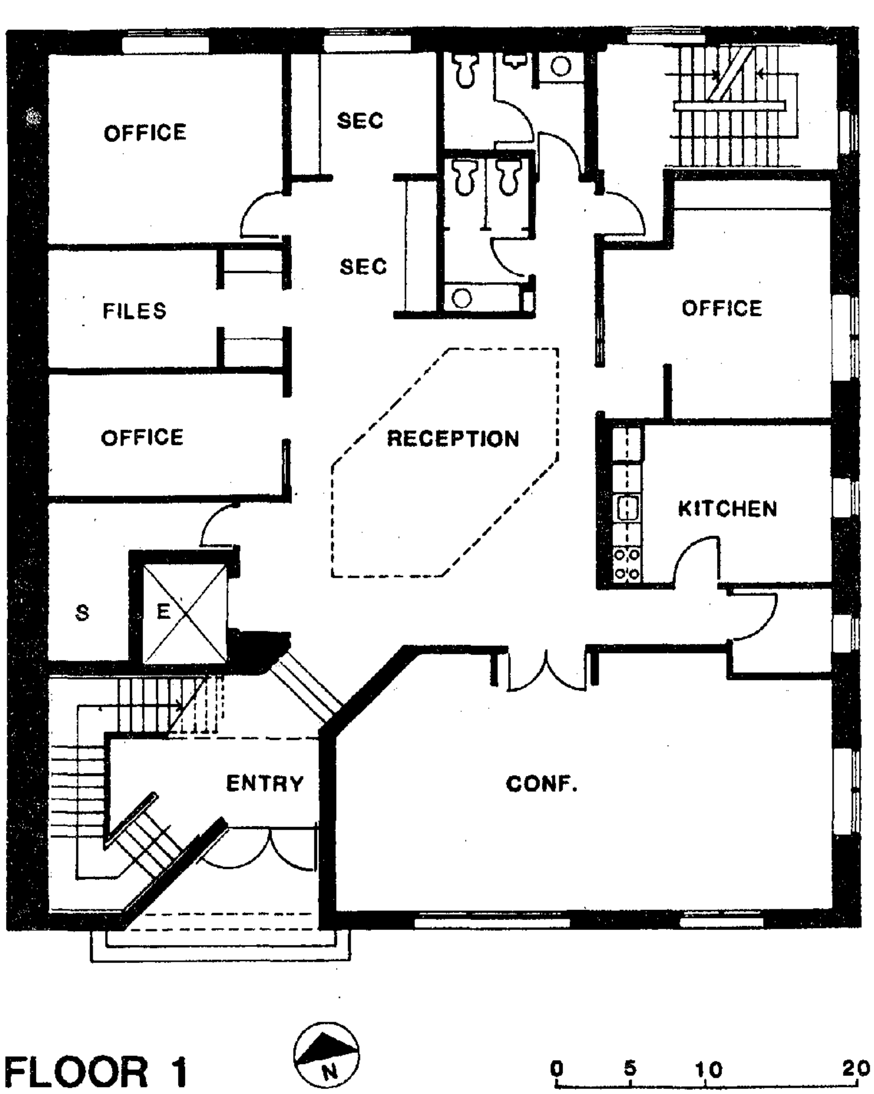


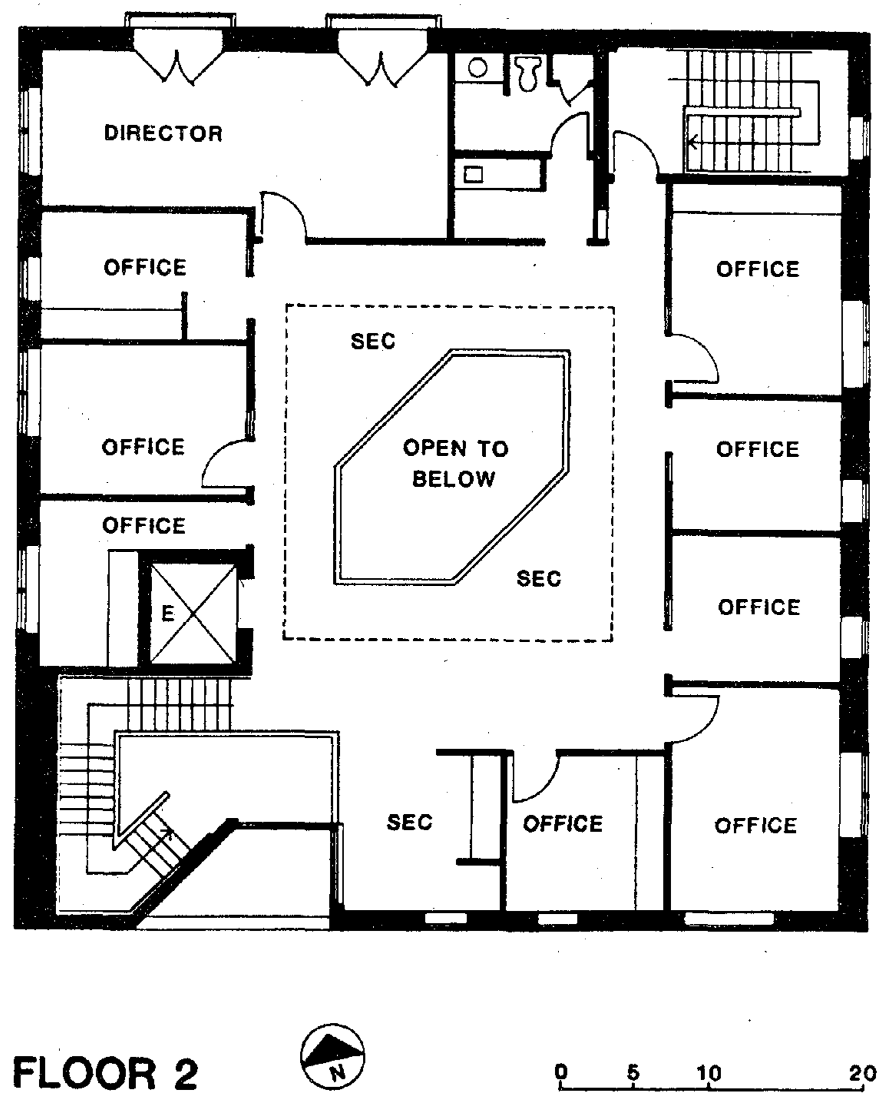
SITE PLAN



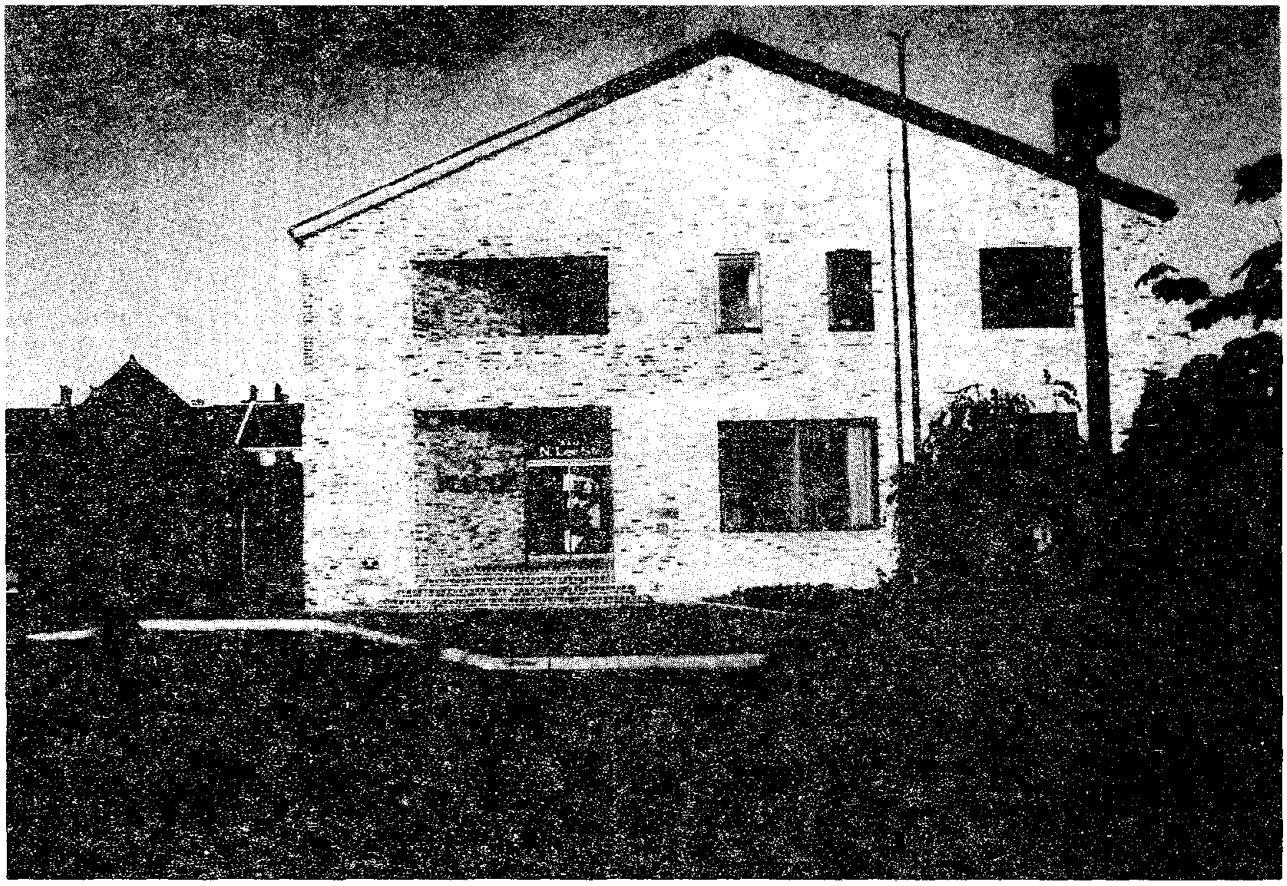
BASEMENT

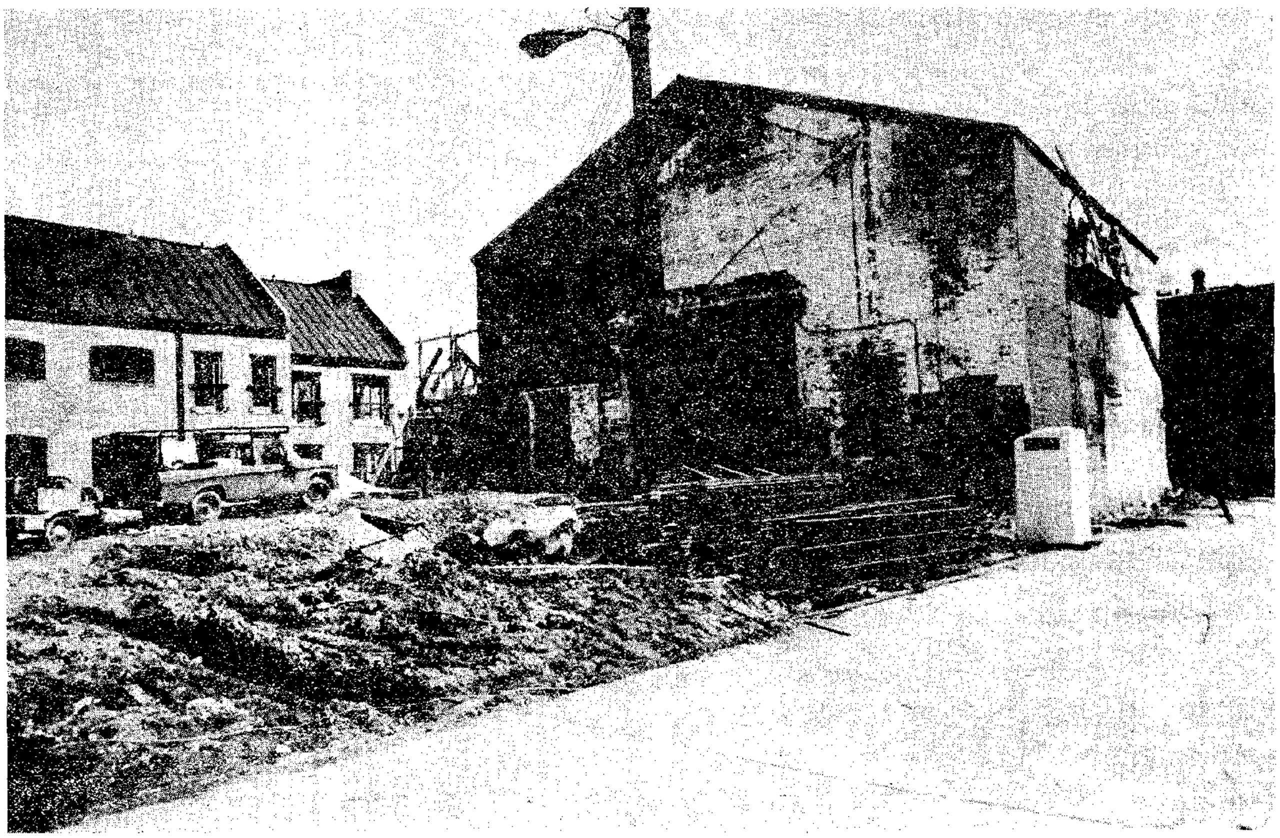


FLOOR 1



FLOOR 2





BUILDING BEFORE RENOVATION

ANALYSIS OF R. C. BUILDINGS SUBJECTED TO LOCALIZED DAMAGE

Brian Hobbs
Department of Civil and Structural Engineering
University of Sheffield England

SUMMARY

This investigation concerns the response of typical reinforced concrete office buildings to localized damage, in the form of the effective removal of one or more columns in a given level. Using model test results as a guide, an analytical approach is developed. Particular features of the analysis are discussed and it is extended to include the influence of infill wall panels on the behaviour of the damaged building frames. The analysis is applied to two sample buildings and results are presented showing how the load factor against collapse varies with the number of columns removed. It is shown that the presence of infill panels considerably increases the ability of structures of this type to withstand localized column damage and it is suggested that the addition of such panels to existing structures may be a suitable means of increasing their ability in this respect.

INTRODUCTION

A common form of structural damage is that consisting of the effective removal of one or more of the columns at a given level in a building. This form of damage must be considered in current British design practice for buildings of more than four storeys (HMSO(1976)). This paper concerns the behaviour of in-situ reinforced concrete structures of the type commonly used for commercial office buildings. The aim of this work is to assess the ability of such structures to survive various amounts of damage and to consider possible methods of increasing this ability in existing structures, and of reducing the costs of repair following the occurrence of such damage. The approach adopted has been to consider two actual modern designs for buildings of this type. Building I has flat plate floors with edge beams, but no internal beams, and Building II has waffle slab floors. Details of typical sections of the floor plans of these two buildings are given in Figure 1. These buildings are considered to be fairly representative of this class of structure and it is hoped that results based on them may provide indications of general trends.

A programme of 24 tests on 1:20 scale models based on the form of Building II has been carried out in order to investigate the modes of failure resulting from various configurations of damage. Full details have been presented elsewhere (Hobbs and Cubison (1979)). After loading to failure the models showed the formation of distinct yield line patterns in the floor slab, the precise form of the pattern being influenced by the reinforcement details as well as by the locations of the damaged columns. An analytical approach based on the yield line method has therefore been developed. Model tests in which infill wall panels were introduced in various positions indicated that such panels could lead to considerable increases

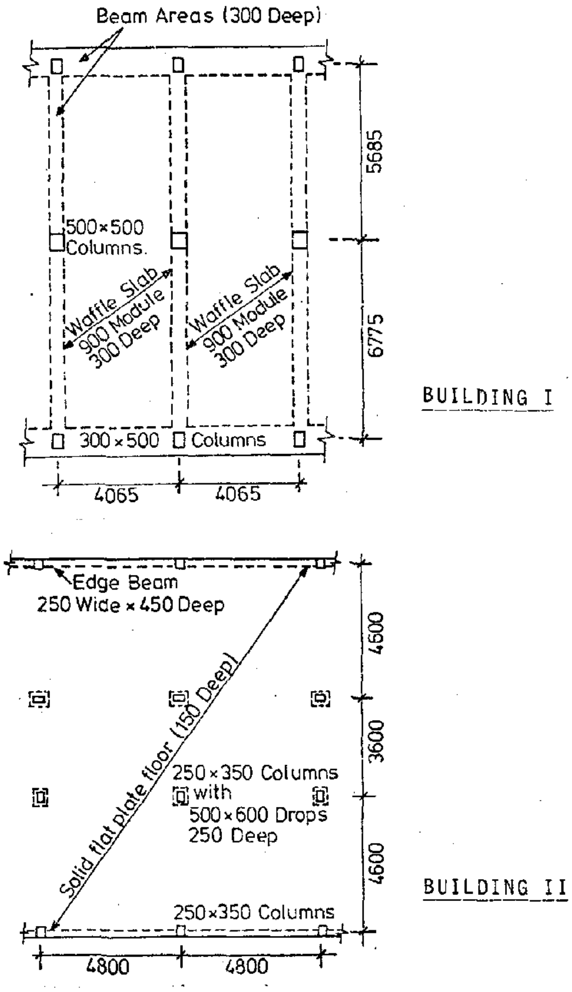


FIGURE 1 - Typical Part Floor Plans

in the strength and stiffness of damage structures. This suggested that the replacement of non-structural partitions or cladding by masonry panels built into the structural frame could provide a useful means of increasing the capacity of existing structures to withstand the effects of localized damage. The analytical approach has therefore been extended to take into account the presence of such infill panels.

METHOD OF ANALYSIS

General

The yield line method due to Johansen (1962) is a well established technique for the analysis and design of reinforced concrete slabs at the ultimate limit state of collapse. The approach may be readily extended to take into account the effect of beams crossed by yield lines (see Wood (1961), Moy (1981)). When used as a design method the resulting slab reinforcement is usually either completely uniform or arranged in uniform bands across the slab. In applying this technique to the analysis of floors which have been designed by other methods due account must be taken of the influence of the resulting reinforcement detailing and curtailment on the pattern of yield lines at failure. This, and other features of the analysis, are described below.

The case of corner damage will be used to illustrate various aspects of the analysis. Similar principles apply to other damage configurations.

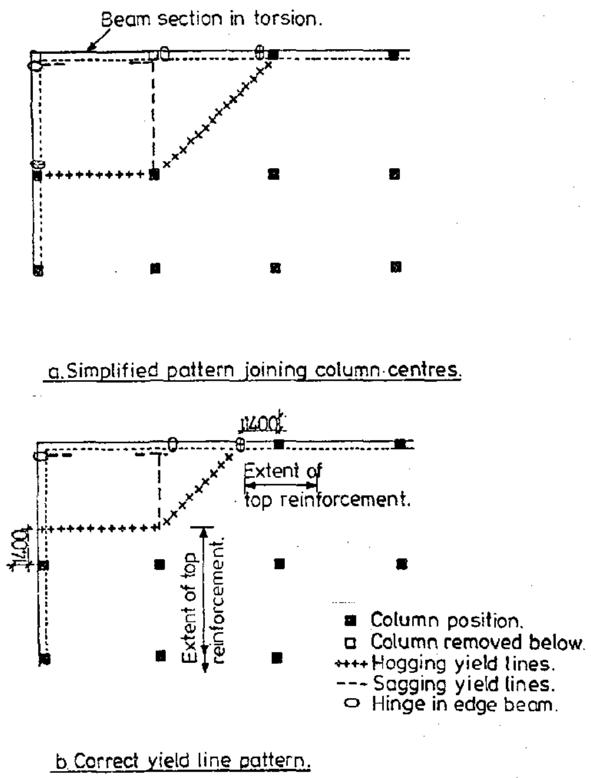


FIGURE 2 - Examples of Yield Line Patterns

Influence of Reinforcement Arrangement

Hogging yield lines are necessary for the formation of collapse mechanisms in floor panel with one or more continuous edges. In slabs with uniform, or near uniform, arrangements of top reinforcement these hogging yield lines form along the column grid lines when the loading is predominantly uniformly disturbed. An example of this type of yield line pattern, for the case of corner damage with two columns removed, is shown in Figure 2a. When top reinforcement is provided only locally in the regions of hogging bending moment, however, and is either omitted altogether or drastically reduced in the sagging regions, the hogging yield lines may form along the lines of reinforcement curtailment, as shown in Figure 2b. This latter type of yield line pattern was observed in many of the model tests and, in order to assess the importance of taking into account the reinforcement arrangement when optimising the yield line pattern some calculations have been carried out for both types of pattern. These showed that use ofthe simplified patterns could overestimate the strength of a damaged building by between 10% and 50%.

Behaviour of Floor/Column Connections

In many cases the detailed behaviour of the connection between the floor structure and the columns must be taken into account. Consider, for instance, the case of corner damage with one column removed. The collapse pattern of this case is illustrated in Figure 3. If the removal of the column below level 1 results in significant damage to the connection at A, then rotation may occur at the connection with relatively little resistance and a conservative estimate of the collapse loading may be obtained by assuming a free hinge at this connection. If, however, the connection remains undamaged, then rotation may occur in one of two ways. These are by the formation of a sagging yield line in the floor adjacent to the column or by the formation of a hinge in the column just above the floor level. At the corresponding junction on level 2, labelled B in Figure 3, rotation will require the formation of either sagging yield lines similar to those in the floor at level 1 or column hinges both above and below the floor level. Calculations for the two buildings considered herein showed that the floor yield line mode is weaker than the column hinge mode, even at level 1, so that failure would occur with identical yield line patterns in all floors and without the formation of column hinges.

Effect of Infill Wall Panels

The influence of infill wall panels on the behaviour of building frames subjected to horizontal racking loads has been the subject of numerous investigations. These have recently been reviewed by Barua and Mallick (1980). It has been found that the presence of infill panels may lead to considerable increases in the racking strength and stiffness of a structural frame. The forces experienced by an infilled frame panel in a building subjected to column damage are illustrated in Figure 4. The case of a panel subjected to a racking load is shown for comparison and it may be seen that the situation of the panel in the damaged building is analogous to that of the racking load situation rotated through 90^{o}.

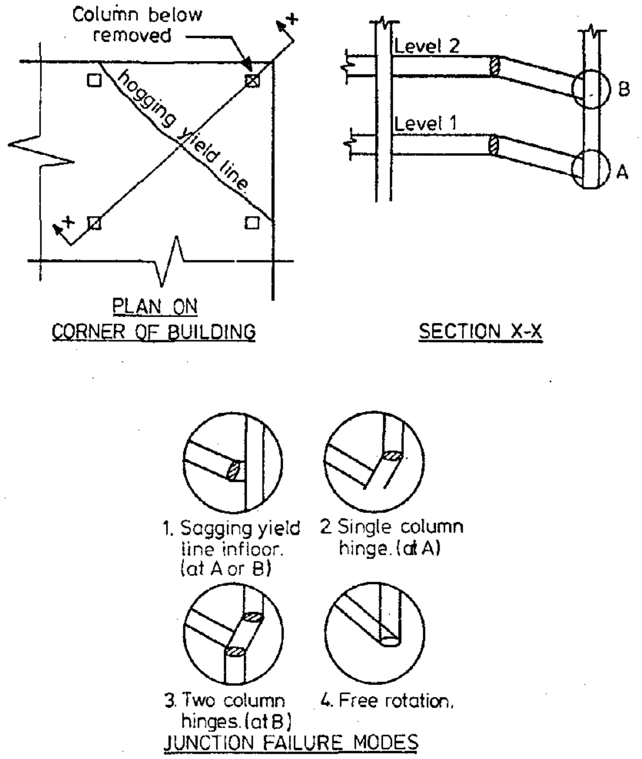


FIGURE 3 - Behaviour of Floor/Column Junctions

Wood (1978) has developed an analytical approach based on the principles of plastic analysis and provides a method of estimating the plastic collapse load of an infilled frame. This approach is thus compatible with the yield line analysis used for the floor slabs and has therefore been adopted for these calculations. For most of the calculations the panels have been assumed to have a design ultimate compressive strength of 3 N/mm^2. For masonry walls designed according to BS 5628 (BSI(1978)), this strength would result from the use of the normal partial safety factor for material strength of 3.5 and walls constructed from standard bricks of 50 N/mm^2 strength, or normal solid concrete blocks of 16 N/mm^2 strength, set in a 1:1:6, cement:lime:sand mortar. In order to assess the influence of masonry strength on the calculated collapse load, some calculations have been performed for the full range of likely panel strengths. These showed that for all except very low panel strengths (<0.7 N/mm^2), failure should occur in the diagonal compression mode designated DC by Wood. The application of the analysis depends upon the introduction of an empirically based penalty factor in order to allow for the difference between the actual behaviour of a masonry infill panel and its theoretical ideally plastic behaviour. It is in the assessment of these penalty factors that the greatest difficulty in the application of this approach lies. Most of the test results used by Wood are, however, for panels failing in the diagonal compression mode. The penalty factros quoted should therefore be reasonably reliable for the panels analysed herein.

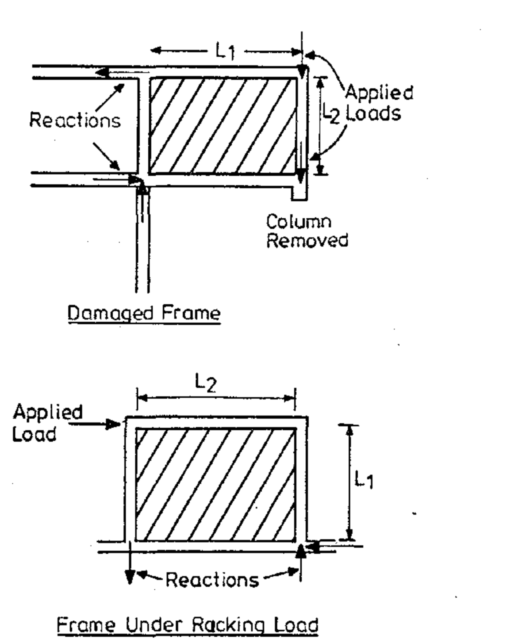


FIGURE 4 - Forces on Infilled Frame Panels

In calculating the failure moads of the infill panels no account has been taken of the effect of adjacent panels on the behaviour of the critical panel. In practice, when a panel is surrounded by adjacent panels it can only fail in the enforced shear mode, designated S by Wood, and this leads to a slightly higher theoretical collapse load. In many situations the failing panels will be bounded by other panels only on some sides, so the mode of failure will be a composite DC/S mode. In this case use of the mode appropriate to an isolated panel gives slightly conservative results and comparison with the model tests show that the errors are small.

One possible mode of failure which has not been considered in detail is that of shear in the columns or beams surrounding the infill panel. Where there are adjacent infills this mode of failure will be prevented, but where a compression corner of an infill is restrained only by the beam or floor slab and the column there is a possibility of shear failure in one of these elements before the full panel load is reached. The actual distribution of shear forces at the corner of a panel is very difficult to determine since it is influenced by the frictional characteristics of the masonry/concrete interface. As noted by Wood (1978), this aspect of the behaviour of infilled frames needs further detailed investigation. Approximate calculations have been performed, however, and these show that, for the buildings considered herein, shear failure is unlikely to be a problem with low and medium strength infills, but that if very high strengths are used there is a possibility that premature shear failure may occur. It is suggested, therefore, that reliance should not be placed on such high panel strengths until further investigations have taken place.

RESULTS AND DISCUSSION

Frames without Infill Panels

The two sample prototype buildings have been analysed for the effects damage occurring in three basic configurations. For each case the number of columns damaged has been increased gradually and the overall load factor against collapse, γ, has been determined at each stage. The value of γ is based on a total load consisting of the dead load of the structure and one third of the design floor live load (HMSO (1976)). The results and the damage configurations considered are presented in Figure 5. The analyses have been performed for a single floor level on the assumption that all floors are similar and are subjected to similar loading.

Despite the differences in their form of construction, it is seen from Figure 5, that the results for the two structures follow a similar pattern, although the load factors for Building II are generally higher than those for Building I. This may be a result of the higher self weight of the floors in Building I. The required load factor for the damaged condition is 1.05 (HMSO(1976)) and it is seen that, if only one column is removed, the only case yielding a load factor lower than this is for the removal of the corner column together with damage of the floor/column junction. For the edge damage mode the removal of two columns is necessary before the load factor drops below 1.05 and in the centre damage mode three columns must be removed.

These results demonstrate a considerable ability of this type of structure to "bridge over" damaged columns, especially when they are situated within the building, and highlights the corner column as being the most critical. They also show that, for in-situ concrete floors, it is not necessary to rely upon the concept of catenary action developed for two dimensional frames (Wilford and Yu (1973)). The deflection associated with the formation of the yield line patterns considered herein are much smaller than those associated with the development of catenary action. This may have important repercussions on the resulting costs of repair for the structure.

For cases involving floor/column junction damage these results provide a conservative estimate of γ for the complete structure since it is possible that only the joints at the top of the columns actually removed may be damaged, while the joints in floors above this level will remain undamaged. Thus a realistic estimate of the overall load factor will lie somewhere between the values for the two cases presented in Figure 5. The larger the number of storeys above the damage level, the closer will the overall load factor approach the value for undamaged joints.

Effect of Infill Panels

The effect of infill panels will depend both upon the position of the panels in the building plan and also upon the number of storeys of infill above the damage level. Results for the common case of infill along the edge of the building are presented in Figure 6 for the three basic modes of damage and assuming either 1 or 3 storeys of

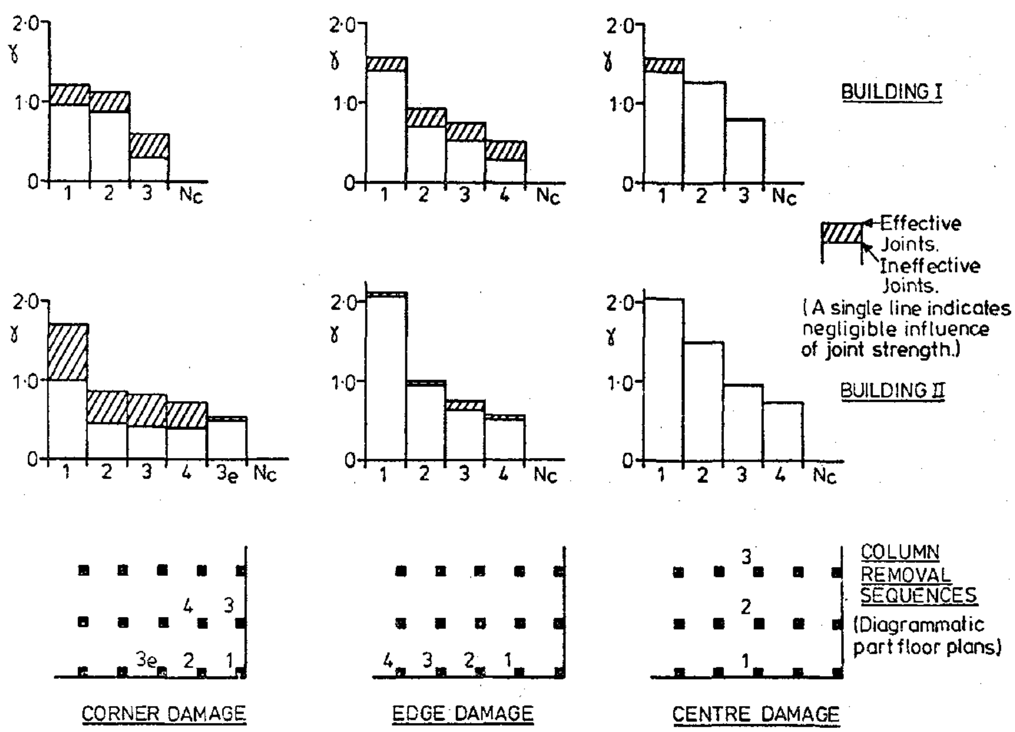


FIGURE 5 - Results of Buildings without Infill Panels

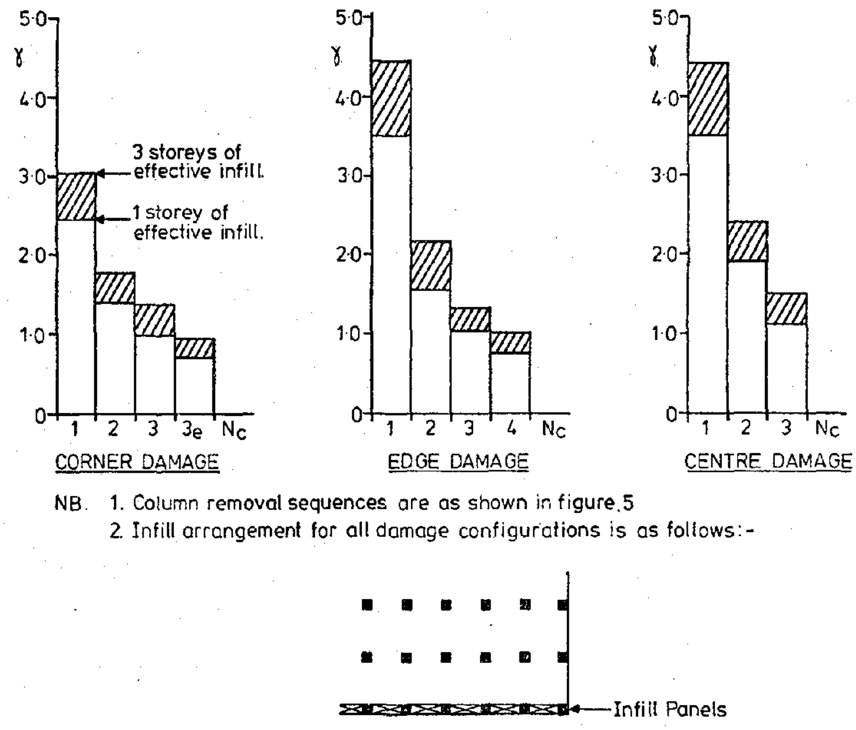


FIGURE 6 - Results for Building II with Infill Panels

infill above the damage level. In assessing the number of storeys of effective infill panels above the damage level, it must be remembered that the column/floor joints must be intact at all four corners of an infill panel in order for it to be fully effective.

It is seen that the presence of infill panels leads to considerable increases in strength and that, even for the corner damage mode, it is necessary for at least three columns to be effectively removed before the load factor reaches the critical level. Other infill panel arrangements could be devised so that a greater number of panels is involved in each collapse mechanism, with the correspondingly greater increase in load factor for that damage pattern.

The presence of infill panels will also lead to a considerable increase in the stiffness of a damaged structure. This is illustrated in Figure 7, which shows experimentally determined load/deflection curves for two modes of damage. This, again, could have a considerable influence on the cost of repair for a damaged structure.

Strengthening Existing Structures

It is clear that the inclusion of infill panels in a structure may lead to considerable increases in strength and stiffness. The replacement of non-structural partitions by structural masonry may provide a suitable means of increasing the ability of an existing structure to survive structural damage of the type considered herein. Even low strength infills may be of considerable assistance in this respect and the use of lightweight concrete blockwork, which would

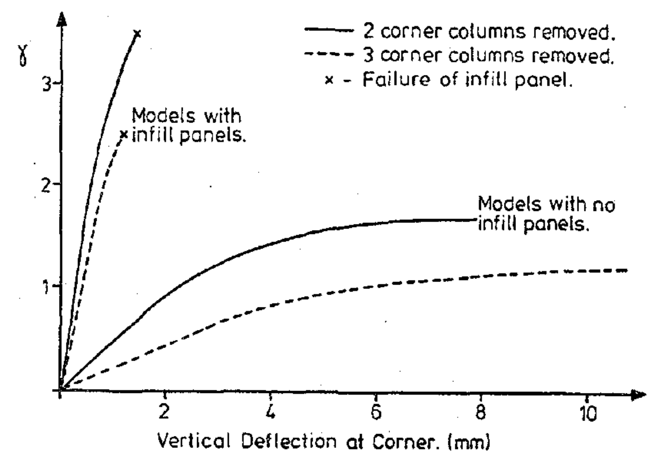


FIGURE 7 - Influence of Infill Panels on Deflections

minimise any problem due to additional vertical loads on columns and foundations, may be the most appropriate solution. Reliance should not, as yet, be placed on high strength infill panels since the resulting shear forces on beams and columns may lead to failure of these elements.

If infill panels are undesirable, then an alternative approach may be to provide additional hogging reinforcement in the floors. This could perhaps take the form of mesh reinforcement laid in a new structural screed, or of steel plates bonded by epoxy adhesive. This reinforcement should be placed so as to intercept the hogging yield lines in the appropriate collpase patterns. The provision of top reinforcement in any areas of floor slab without existing top reinforcement would be of particular value in preventing the formation of hogging yield lines along the lines of curtailment of the top reinforcement.

CONCLUSIONS

This investigation has been concerned with analysing the response of reinforced concrete buildings with monolithically cast floors to damage in the form of the effective removal of one or more columns at a particular level in the building. The main conclusions may be summarised as follows:-

1. An analysis based on the yield line approach reflects the observed behaviour of model structures.

2. In applying a yield line analysis, particular attention must be paid to the influence of reinforcement distribution and curtailment on the resulting critical yield line pattern.

3. The analysis may be extended to include an assessment of the influence of infill wall panels by adapting an existing plastic analysis for shear wall panels.

4. Buildings of this type have a considerable ability to 'bridge over' damaged columns. If only one column is damaged, then it is only when this is situated at the corner of the building and when the floor/column connection is also damaged, that the resulting load factor is likely to be unsatisfactory according to current criteria.

5. The presence of even relatively low strength masonry infill panels leads to considerable increases in the strength and stiffness of a damaged building. The addition of such infill panels to existing structures may provide a suitable means of strengthening them to resist damage of this type and also fo considerably increasing the chances of economic repair subsequent to the occurrence of damage.

ACKNOWLEDGEMENT

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REHABILITATION OF A HISTORIC MANHATTAN BUILDING CHICAGO PROBLEMS & IMPACT ON ENVIRONMENT

D. K. Gupta
President

Manu B. Patel
Vice President

DKI Group Engineers, INC
Calumet City, Illinois 60409

SUMMARY

As a part of a rehabilitation project of 4 buildings in the south side of Chicago, a complete investigation was carried out for the Manhattan Building. This building was built almost a hundred years ago, with a metal frame skyscraper which was revolutionary in the architecture of its time. It was the first building to include internal wind-bracing. As a historic building, the main goal of the project was to maintain their values, but to renovate them with modern amenities to make them functional for present needs. This paper presents the essential aspects of the project and includes the following aspects: a) History Inspection Report; b) Feasibility Studies for different uses; c) Environmental impact; d) Design criteria, and e) Solution (mechanical, structural and other types). It was shown that the solution was the most viable and beneficial from development point of view.

INTRODUCTION

Rehabilitation of old and historic structures has become increasingly important as a development alternative because of exhorbitant costs of new construction and because of the recognition of the importance of the preservation of our cultural heritage. Specific problems such as deteriorated building condition, economical conversion of interior space for alternative uses, or conflicting adjacent land uses often affect the feasibility of rehabilitation projects. These problems can be overcome, however, with innovative engineering and site planning and by careful study of the relationship of a project to local development trends. This paper is a result of one such project called "Manhattan Building Rehabilitation"in Chicago area.

HISTORY OF MANHATTAN

The Manhattan was designed by William LeBaron Jenney, inventor in 1884 of the metal-frame skyscraper that revolutionized world architecture. It has a distinctive and charming facade with a variety of big bay windows that catch the light, and was the first tall building to include internal wind-bracing.

The works of William LeBaron Jenney predate the Chicago school, but it is here that such noted Chicago school architects as Daniel Burnham, Martin Roche, William Holabird, and Louis Sullivan found their training. Jenney, trained as an engineer in the United States and France developed many of the basic principles of the Chicago school of architecture. His design for the Leiter Building, 1879, represented an intermediate step between the then prevalent masonry construction method and true steel frame construction. In the Home Insurance Building of 1884, Jenney further explored the possibilities of complete frame construction.

In the Manhattan Building, Jenney's system of internal framing was fully implemented. The entire height of the sixteen story Manhattan Building is supported by its steel and iron fram ; wind-bracing was provided by a portal system, but now depends upon the load-bearing walls at the north and south ends of the building for rigidity. Contrary to previously published scholarship, this study has found that the north and south walls of the building are supported on grillages, which are original wind-bracing have been removed, making that system of wind-bracing inoperable. This does not detract from the fact that, upon its completion, the sixteen story Manhattan Building was triumph of engineering.

The structural accomplishments of the Manhattan were accompanied by an attempt to produce a design related to functional considerations. The narrowness of the block of buildings between Dearborn and Plymouth afforded the architect an opportunity to design a building with two main facades and an opportunity to give natural light to every room. While the basis for much of the facade ornament lies in Romanesque architecture, the first floor of the Manhattan almost proudly displays bolted-on cast iron panels separating the shop front windows; a very bold expression of modern materials and methods in its day for a building of this type.

Rusticated grey granite, sheathing the second and third stories, provides a powerful image from the street. The notable visual element of the Manhattan Building is the variety and arrangement of the bay windows that project from the east and west facades. At the time the building was being designed, two twelve story building were being constructed directly across Dearborn Street. Jenney decided to design bay windows to admit light to the lower floors that might otherwise have been blocked by the building across the street. This rationale is still valid today. Above the belting course at the third floor ceiling level, the building breaks into a series of rounded and polygonal bays in an A-A-A-B-B-B-A-A-A- pattern. Portions of this design drop off after five floors, but the polygonal bays of the B pattern continue to the eleventh floor. Ornamental terra cotta work sheaths the structure that support these bay windows. Above the ninth floor, a belting course adds horizontal emphasis; at the twelth floor the piers seem to end in classical column capitals which in turn support a cornice. Some previously published scholarship suggests that the top four floors of the Manhattan were a subsequent ad to the building, but this study has found no evidence to support this, either in examining the original building construction or in photographs taken shortly following construction. The Manhattan Building is nine bays wide; the south and north bays originally rose to a height on only nine floors and buttressed the central sixteen story portion of the building. This setback element of the Manhattan's design was also used by Adler and Sullivan, and others in subsequent Chicago school designs. In addition to eliminating excess load on the party walls to the north and south, this setback allows for increased light and ventilation to interior areas above the ninth floor. The exterior of the Manhattan Building reveals two small single bay additions above the ninth floor, on both north and south ends. While the parapets have been rebuilt, the overall design of the building remains largely intact, except for alterations to the ground floor necessitated by the explosion in 1968 of a building across the street. Besides blowing out windows in both the Manhattan and the Old Colony, this explosion extensively damaged the ground floor iron work on the Manhattan.

When originally built, the Manhattan Building was one element in a long block stretching between Van Buren Street on the north and Harrison on the south. The Manhattan, originally buttressed by two eight story buildings, was designed as a printing loft structure like its neighbors to the south. Since the construction (in the late 1940's) of the Congress Street Expressway, the Manhattan has lost its association with the printing house district. The narrowness of the blocks in this area made it an ideal location for the industry. Its proximity to the Dearborn Street Station, at the intersection of Dearborn with Polk, made it easier for the printers to receive supplies and to ship their goods. The longitudinality of the building allowed an ideal set-up for presses. The Manhattan has since been used for such activities as printing and other light manufacturing interspersed with offices of printers and transshippers.

Today the Manhattan still provides the same sort of office/loft uses originally envisioned by the architect. Except for modernized portions of the building used by its present prime tenant, the Manhattan Building retains many of its spaces in original, if somewhat worn, condition.

FEASIBILITY OF MANHATTAN FOR VARIOUS USES

As a part of rehabilitation, several alternates were investigated to reuse the building for its occupancy. A brief report on this study follows:

VERTICAL SHOPPING OR OTHER RETAIL USE: The greatest impediments to adaptive reuse of the study buildings for vertical shopping are their design and floor area characteristics. The arrow thin slab design in this building makes it unattractive as branches of the major retail department stores operating in the Chicago market. Other aspect to be looked into is the need for parking garage in the proximity of shopping.

Developers of vertical shopping centers customarily provide an attached parking garage, preferably with entrances to the shopping arcades at each parking level. Manhattan Building has an adjacent site which could be utilized for an attached parking garage. The Building embodies another deterrent to adaptive reuse of the four subject buildings for vertical shopping: poor location. It is located on the south edge of the Loop's commercial office concentration. Much of the shopping activity in the Loop is generated by office workers during a lunch hour or immediately after work. The major department stores at the south end of State Street, Sears and Goldblatts, attract fewer noon hour and after work shoppers than the Marshall Field, Carson Pirie Scott, Montgomery Ward and Wiebold department stores further north on State Street and in closer proximity to the greatest concentrations of office workers. The same locational disadvantages that affect the Sears and Goldblatt stores would also affect any new retail shopping in Manhattan building.

HOTEL: Chicago is the largest convention city in the United States, having 2.3 million convention visitors in 1977, who spent $509 million and accounted for approximately seventy percent of the occupants at Loop and North Michigan Avenue hotels.

The location of the Manhattan building on the south edge of the Loop makes them unattractive for hotel use. The other hotels in the area are the Pick Congress, Blackstone and Conrad Hilton hotels and the Loop hotel market could support additional rooms. However conversion of the building in this study to hotel use would be economically infeasible. The number of rooms that could be constructed in any of the buildings would be less than the desirable number from the point of view of a hotel developer. In none of the buildings would there be adequate space for the banquet facilities, meeting rooms, and other public spaces that help attract conventioneers and businessmen to Chicago. Adjacent parking would be a problem for adaptive reuse for hotel purposes just as it was a problem in the consideration of vertical shopping uses.

The cost per room of new hotel construction in Chicago is approximately sixty-five thousand to seventy-five thousand dollars. Unlike adaptive reuse for other purposes, adaptive reuse of commercial structures for hotel purposes could result in an even higher cost per room than new construction. The interior of any of the four buildings in this study would need to be completely gutted for conversion to hotel use. Routing each room's plumbing and heating service through the exsting floor structure would significantly increase the cost of providing those services. The result would most likely be a more expensive hotel to build, and less attractive to potential hotel customers because of the unavailability of meeting rooms and banquet facilities.

EDUCATIONAL: When the Chicago City College system was considering sites for the new Loop College, a proposal to incorporate the Old Colony Building as part of the new facility on State Street was submitted by the architectural firm of Metz, Train, Olson and Youngren for the Landmarks Preservation Council and Service. The City College board rejected that proposal and opted for a totally new structure on two blocks fronting State Street. Other Colleges, including La Salle Extension University presently occupying about 40% of the Manhattan Building, have great uncertai about their expansion or upgrading. Thus, they are unlikely to be a source of new tenants or owner for the subject building.

GOVERNMENTAL: One possible public role would be acquisition or lease of the building for government offices. However, present prospects for such use are slight. The federal government has constructed two new office buildings immediately north of the Monadnock Building on Dearborn Street within the last twelve years. It has consolidated its offices into those new buildings. The federal government now occupies approximately 412,000 square feet of office space in downtown Chicago and its need for additional space is limited. The City of Chicago occupies approximately 494,000 square feet of office space, the bulk of it in the historic City Hall (presently undergoing extensive renovation) and the Richard J. Daley Civic Center on Dearborn Street. It, too, has little present need for additional space.

RESIDENTIAL: The interior plan of the building does not land to easy conversion of open commercial loft space for open residential loft living economically. Demolition of interior walls to create open loft-type space would be unnecessarily expensive. The original office configuration of the study buildings may be better suited to conversion to conventional one, two or three bedroom apartments.

Because the Manhattan Building is closest to the Printing House Row and Dearborn Park developments and is not adjacent to the elevated tracks, it has the best potential for conversion to a conventional apartment building. As indicated in the full estate and economic feasibility analysis, the Manhattan Building has been analyzed in this study as an example of conversion to residential use. The Manhattan Building has advantages for residential use over the other study building in addition to its proximity to Printing House Row. The half block which separates the elevated railroad tracks from the Manhattan Building makes the train noise unnoticeable. Only the Mantattan Building has an adjacent lot for attached parking. Attached parking is absolutely essential to the marketability of conventional apartment units. Figure 1 shows details of various landmarks in the area in the Loop area where the Manhattan building is located.

CONDITION SURVEYS

Structural, mechanical, and electrical engineers collaborated in assessing the condition of the building structure and related building systems, and an initial determination of the levels of structural modification and system replacement necessary to restore the structure was made, including cost estimates Fire and Safety, and Real Estate report. This report is presented in this section.

ARCHITECTURAL REPORT

The building is sixteen stories tall (not including the basement) and has a brick exterior, a stone foundation, and a flat roof. The north and south bays are ten stories (the tenth floor was added subsequent to original construction); and the central portion of the building is sixteen stories. The construction is steel frame with cast iron columns, and tile arches for infill. Partitions are plaster on tile.

EXTERIOR: The Dearborn Street elevation has pressed tan face brick, terra cotta trim, and tile coping. It is dark gray from the years of weathering. There are three-sided bay windows at the center of the facade on the fifth through eleventh floors, and circular bays (three at either end) at fourth through eighth floors. The three center bays have elaborately sculpted soffits. Window openings at the nineth floor (in added bays at either end) are arched, windows wood sash double-hung, with washing lugs. The exterior is rusticated granite with terra cotta mullions at the second and third floors. The first floor has the original ornament steel storefronts, with a single wood and glass door in each bay; some have angled recesses and transoms and the balance are flush. There is no commercial development on the first floor except for the small fast-food store, all glass has been painted. The entire first story is deteriorated and shabby. There are various bulkhead materials.

The main entrance at the center bay has been refaced with pitted corrugated aluminum at the columns and the spandrel, and has new soffits and recessed incandecent lighting. There is a varnished wood and glass revolving door with a pair of varnished wood and glass doors at either side. The facade of floors two through sixteen remains in original, if neglected, condition in contrast to the more abused ground floor facade.

The south elevation formerly abutted an adjacent building. The ground through eighth floors are common brick, in rough condition, with remnants of demolition, and have been painted. The ninth and tenth floors are structural tile with tile coping, and have double-hung window sash. The Plymouth Court elevation is pressed brick, with terra cotta trim, octagonal bays at either end at the fourth through either floors, and three octagonal bays at the center at the fourth through eleventh floors. There are arched window openings at the fifteenth floor, with double-hung wood sash with window washer lugs. The second through sixteenth floors have two wrought iron fire escapes with roof ladders that are counterweighted at the second floor for access to grade. This elevation is also gray from dirt.

The north elevation is flush with the adjacent building up to the tenth floor. Above the tenth floor is structural tile with brick and terra cotta cornice, and double-hung wood sash.

There are four connections to the building to the north. These are the fourth, second, and first floors, and the basement. The second floor opening was created in approximately 1967, the fourth was completed in 1963, the basement three years ago, and the first floor seems to be the oldest; there is no recollection of how long ago this was constructed. An investigation was made of floor alignments to the building to the north, and it appears they connect fairly well at all levels up to the eighth. The Manhattan Building supplies heat, hot and cold water and fire protection to the adjacent buildings to the north. The fire pump (standpipe) also serves both buildings.

BASEMENT: Prior to 1947 access to the basement was by a manhole only, located in the Plymouth Court sidewalk. A stair installed at that time is open from the loading area to the basement. The basement has a concrete floor, unfinished and semi-finished stone and structural tile walls, structural tile ceiling approximately eight feet high, with exposed pipes, and exposed bare bulb lighting. Some pipes are below head height. All elevator equipment is located in the basement. There are two boilers: a Kewanee type 'C' boiler with oil burner, and outside coil water heater, and a smaller boiler with a Rockwell burner, outside coil water heater and circulating pump. Other basement equipment consists of a hot water tank, Chicago Pump, twin sewer ejector, and two Bock high-recovery water heaters. A carpentry shop is located off an employee washroom. A counterweighted fire door protects connection to the north adjacent building occupied by LaSalle Extension University. There is an 8,000 gallon oil storage tand with brick and tile enclosure filled with sand, as required by code.

In the northeast corner of the basement there is a stair leading out to the sidewalk. It has a makeshift door with a grille for boiler fresh air. Many different levels are in the basement and it extends out under the Dearborn Street sidewalk.

TYPICAL FLOOR: The typical corridor has the original marble, four foot high marble dado, fire escape signs, wood doors with obscure glass, cast brass mail chute and initialled hardware, and wood frames, operable transoms with transluscent glass. There are two telephone pull boxes and one electrical closet on each floor, along with a standpipe system with a fire hose opposite the electrical closet and adjacent to the stair. A single open marble and cast metal stair exists between the lobby and the sixteenth floor. The corridor ceiling (recessed 2x4 fluorescent lighting and "modern" dropped acoustical tile ceiling with exposed grid) is continued on into the stair. Above the sixteenth floor the stair is open steel construction to the roof, adjacent to open elevator shafts (nopartition)

OTHER FLOORS: The sixteenth floor is in nearly original condition and is presently being used for building storage and service functions. The original open elevator cages are still intact.

The ceiling is 10 feet high and plastered, with exposed bare bulb incandescent lighting. There is some exposed piping.

The fifteenth floor was largely rebuilt following fire damage, with more modern partitioning,etc. The tenth, eleventh, twelth, thirteenth, and fourteenth floors are typical.

The added north and south bays have triple arched openings separating them from the main bulk of the building. There are exposed anchors at top and bottom of columns where diagonal wind bracing was once attached. Some wind bracing survives in one partition at the north end of the building. The ceilings have been dropped with acoustical tile with continuous stripmounted fluorescent lighting four feet on center. Major toilet facilities are located on this floor. The men's washroom was remodeled in 1947 and has a new terrazzo floor, with plastered walls and ceiling. The women's washroom is epuipped with seven toilets and two lavatories. The finishes are the same as the men's washroom.

The eighth floor north is presently being used by LaSalle Extension University as a library, and has a partial acoustical tile ceiling and fluorescent lighting. In room 827 one can see the gusset members which evidently replaced the turnbuckle wind bracing which formerly went top to bottom diagonally between columns.

The seventh floor corridor is narrower than other floors at the elevator entrances. There is a differential of approximately three inches in floor levels at the seventh floor, roughly midway between the north and south walls.

Wind bracing is visible in the center bay only on the sixth floor; it has been removed from other bays. Approximately ten wet columns were observed on this floor, which is probably the case on all floors. This floor is used as a warehouse and is very abused where lift trucks drive. There is a women's lounge (with resilient tile floors, and ceiling finished as in the corridors) on the fifth floor. A washroom with ten toilets and three lavatories is located behind the lounge. The fourth floor has been fully "modernized". The men's toilet has been remodeled. The space has plaster and drywall partitions, resilient tile floor covering, dropped ceilings eight feet high with exposed metal grid, recessed 2 x 4 fluorescent lighting fixtures, circular air conditioning diffusers, and a public address system. There is a ramp to the adjacent building to the north, with a counterweighted fire door protecting the opening. The mechanical room contains thirty ton Westinghouse packaged air conditioning unit which provides chilled and heated air, supplimented by perimeter radiation in severe weather.

The corridors on the second and third floors have the original mosaic marble chip floors (in fair condition). The second floor has been covered with a skim coat of latex, and asphalt flcortile.

The north part of the first floor is a semi-finished commercial space with linoleum floor covering over wood, plastered walls and approximately sixteen feet high ceiling (fourteen-and-a-half feet to the beams). Lighting is surface-mounted, and dropped fluorescent. There is an entrance to the building to the north protected with a counterweighted fire door. There are two enclosed washrooms. In the south part the space is semi-finished, with dropped fiber block ceiling, exposed bulb lighting, central air conditing (equipment has been removed), and there are assorted finishes in fairly poor condition. The food shop north of the main entrance has resilient tile floor covering, paneled and drywalled walls, fiber block ceiling, and fluorescent lighting. The stairwell is enclosed at the first floor, with a pair of fire doors with door checks and panic hardware.

The lobby has a marble floor and walls are surfaced with various materials over plaster. The drywall ceiling is channeled for indirect fluorescent lighting fixtures. There are aluminum and glass doors with checks at the entrance to each store (one south, one north).

TYPICAL OFFICE INTERIOR: Interior column spacing is sixteen feet on centers north to south, (with some variation at both ends) and thirteen feet on center east and west (with slight variations). Most tenant spaces have original plaster ceilings, with assorted incandescent and fluorescent lighting, but some have dropped acoustical ceilings with exposed grid and recessed 2 x 4 fluorescent lighting. Most floors are asphalt tile over the original wood, over magnesite underlayment fill, and sleepers laid on tile arch construction. There is a steam pipe at every exterior column line, indicating a one-pipe system, with radiators below all windows. Many suites have washclosets with wood panel doors, a marble apron lavatory, and incandescent lighting over the medicine cabinet. Numerous vaults are in the building, with plastered walls and ceilings, incandescent lights and steel doors.

ELEVATORS: There are five centrally located manually operated 1890 elevators, with one freight elevator having a capacity of 2,000 lbs. The Manhattan Building elevators are operated by a water hydraulic/cable suspension system. Equipment involved consists of a piston-cylinder unit mounted horizontally in the basement of the building.

The water hydraulic system consists of an open reservoir where water is deposited as the cylinder is emptied and the cab descends. From the reservoir water is pumped to pressure tanks, where an air compressor maintains a 150 pound head of pressure. To make the cab ascend, water from the pressurized tanks is introduced into the cylinder.

Very simply put, when the elevator ascends, a valve is opened to fill the cylinder with pressurized water, and the piston/connecting rod assembly is extended, wrapped around the two groups of sheaves. When the elevator descends, a valve is opened which allows the water within the cylinder to empty into the reservoir tank. The piston/connecting rod assembly is retracted; the twelve passes of cable shortened, thus providing cable length to lower the cab.

In 1947 power for elevators was converted by Gallaher & Speck from high pressure steam hydraulic to water hydraulic, eliminating the need for high pressure steam boilers. Because the equipment is obsolete, one unit has been dismantled to provide parts for the others and is not in service.

Elevator service does not extend to the sixteenth floor except on an emergency basis. The elevators are equipped with a mechanical call register and have exposed electrical conduits. Original steel cage elevator doors are missing on all floors but the sixteenth. The elevators were enclosed in 1963 with plaster on metal lath, tied to the exsting original cages.

TOILETS: Men's toilet facilities are located on the second (not in service), fourth (modernized), sixth, twelth, and fourteenth floors. They have raised marble floors, marble dados, exposed piping, slop sinks, and wall-hung urinals with Chicago flush valves. The twelth floor and below have standing urinals with flush valves.

ROOF: The roof is asphalt with galvanized metal flashings and tile copings. Plymouth Court frontage has a cut stone coping. There is a concrete block elevator penthouse with gutters and downspouts of sheet metal. The main roof appears to be in good condition. It has a relatively steep pitch to the center of the building. with one roof drain each side of the penthouse. The roof and penthouse were rebuilt, following a fire on the fifteenth floor in about 1963. The parapet was reconstructed and the roof resurfaced approximately two years ago. Following the fire, parapet height was reduced by seven courses of brick to eliminate deteriorated masonry, and was capped, using stone to replace the terra cotta on the Plymouth Court facade.

STRUCTURAL REPORT

The Manhattan is a sixteen story skyscraper of skeleton construction using steel beams and cast iron columns. Wind loads are accommodated by masonry shear walls and partly by cross-bracing. The structural system is set on spread footings, with beam and rail grillages.

No evidence of any problems related to structural inadequacy or deterioration was observed. This is true in spite of the absence of steel rod cross-bracing, seemingly provided for in the original design.

This bracing is shown in the original drawings. Brackets which are still visible project from the cast iron columns and were obviously intended to be used as attachments for the bracing rod as can be seen. The building manager stated that rod bracing in two bays of the fourth floor were removed in 1968. No bracing was observed, although in many places it might still be in place, covered by walls. There are many areas throughout the building where this bracing has been removed. Especially important is the fact that the bracing has been completely removed in lower stories, thus making any cross-bracing above completely ineffective.

It is possible that these bracing rods were intended to be only temporary bracing during the erection of the structural frame. If the rods above the first floor were three-quarters of an inch in diameter, (as the drawings show for the basement) they were completely inadequate to provide the lateral bracing required for the wind loads on the completed building. Therefore it is possible that the designer's intent was that the north and south exterior masonry walls acts as shear walls to provide lateral support. In fact, this is what is happening and, therefore, the shear wall bracing is probably adequate without the additional support of the cross-bracing though this has not been substantiated by calculations.

The removal of this cross-bracing was probably done without consulting a structural engineer; such removal or mutilation of structural members could have had serious consequences.

MECHANICAL/ELECTRICAL REPORT

HEATING, VENTILATING AND AIR CONDITIONING

GENERAL: The building is heated by a combination one and two pipe vacuum steam system. Air conditioning is by both individual "window" and central type conditioners.

SPECIFIC NOTES: Two Kewanee Type C low pressure steam boilers installed in 1947 serve both the Manhattan and the adjacent LaSalle Extension University Building. One boiler has sufficient capacity to heat both buildings except on extremely cold days. One boiler is oil fired and the second is combination gas-oil fired. Operation is by pressure stats set to maintain about six psi steam pressure. A masonry stack serves the boilers.

Column-type cast iron radiators at the building perimeter do most of the heating. There are some finned tube convectors in remodeled areas. Control is manual. The heating piping system is two pipe for the ground, second and third floors, and one pipe for the fourth floor and above. Most piping is exposed.

Air conditioning on the second, fourth and fifth floors is done by both water and air cooled central type units. "Window" units serve the other cooled spaces. Generally, ventilation occurs naturally by operable windows.

AREAS FOR CONCERN: Most of the exising heating piping was originally installed in 1891. Some sections have been repaired and/or replaced. This type of maintenance is expected to continue. The aesthetic acceptance of cast iron radiation and exposed piping is open to question. Boiler operation programming based on outside air conditions would save fuel.

The duplex vacuum pump set requires frequent repair and due to its age, parts are difficult to obtain.

PLUMBING

GENERAL: Water service throughout the building is supplied by a duplex house pump system. No house tank is used in this system. Domestic hot water is heated in two gas-fired storage tank type heaters. Roof stormwater and sanitary drainage is piped to city sewers by gravity flow except for basement waste which is pumped. "Wet" columns provide for plumbing needs in various locations.

SPECIFIC NOTES: The building water service is a four inch diameter pipe, fed from a city main in Plymouth Court.

Two house pumps of the same capacity are located in the basement. Each pump has sufficient capacity to serve the building needs. The pumps are used alternately to equalize water. The two gas-fired water heaters are also located in the basement and piped in parallel. Each has its own thermostatic temperature control.

General toilet facilities are located on the second, fifth, and ninth floors. Other individual facilities are scattered throughout the bulding. Fixture ages vary from original to recent installations.

AREAS FOR CONCERN: Like the heating piping, most of the plumbing piping is of the original 1891 construction, and has been subject to repair and/or replacement. This condition can be expected to continue. Plumbing fixtures from the original installation most likely will require replacement due to functional needs and tenant demands.

ELECTRICAL

GENERAL: In 1947 the electrical system was converted from D.C. to a 120/280 volt A.C. system. Each floor has an electrical closet which contains tenant meters and distribution.

SPECIFIC NOTES: The service is 3000 amperes 120/280 volt, 3 phase, 4 wire. The switchboard consists of a 3000 ampere main switch and a distrigution section with 400 ampere fused sections, each serving three floors. Each floor electrical closet contains the tenant meters, along with a cartridge fuse distribution section containing 200 ampere and smaller fuses, which feed portions of a forth circuit plug fuse panel. The plug fuse circuits serve the tenants, mostly through the D.C. system conduit. All tenants are using A.C. except two which still have D.C. equipment. Rectifiers are used for the A.C. to D.C. conversion.

There are many surface mounted electrical conduits and boxes. Active wiring which is not run in conduit still remains in some cases.

Many spaces contain chain suspended globe type incandescent light fixtures. All corridors have 2 x 2 T-bar lay-in ceiling tiles with 4 x 2 four tube fluorescent recessed light fixtures. The exit lights are served from the forty circuit plug fuse general lighting panel on each floor.

Two wood telephone cabinets are located at the ends of the corridor on each floor. The LaSalle Extension University telephones are served from the adjacent LaSalle Extension University Building.

The fourth floor was remodeled in 1974 for office use. A new 2 x 4 T-bar, lay in ceiling with recessed fluorescent lighting was installed, along with battery units for emergency lighting, central air conditioning and new partitions.

The five elevators are hydro-pneumatic operated by means of a horizontal piston and cable reel arrangement. A 10,000 gallon water storage tank, three pumps, three hydro-pneumatic tanks, an air compressor along with the five piston and cable reels are the major items of equipment. Only four of the elevators are in service. The fifth is being used for spare parts because of the difficulty to obtain parts due to their age. The elevator machinery is in the basement.

FIRE AND SAFETY CODE REPORT

SIGNIFICANT CONSTRUCTION FEATURES: The Manhattan Building consists of sixteen stories and a basement. It has approximately 11,000 square feet per floor on the first through tenth, and 8,400 square feet per floor on the eleventh through sixteenth floors. Materials are masonry for the exterior walls, flat tile arch floors topped with concrete and wood flooring (plus resilient floor surfacing), with the floors supported on steel beams that bear on cast iron columns. The columns and beams have been fireproofed with plaster, and reinforced with wire cloth, giving them an estimated fire resistance rating of one hour. The underside of floors is covered by plaster applied directly to the tile and steel runners spanning the beams.

The single open stairway which runs from the ground floor to roof level is located behind the elevator shaft. This stairway is sixty-seven inches wide, with twenty-one steps between floors, without an intermediate landing (six-and-a-half inch risers, twelve-inch treads). Construction is marble on steel supports, plastered beneath (plus dropped acoustic tile ceiling). The existing arrangement requires circling around the elevator shaft on each floor in order to reach the stair continuing to the floor below. The corridor in front of the elevators is about seven feet wide, except on the seventh floor, where width is reduced to about fifty-one inches. Two fire escapes exist on the east facade.

A single four-inch standpipe with one two-and-a-half inch hose valve and one one-and-a-half inch hoseline is connected to a separate valve at each floor, including the penthouse adjacent to the roof access door. This standpipe is supplied by a standard (500 gpm at 125 psi), automatic Peerless fire pump in the basement. Existing is a four inch connection to city mains, with detector check valve. There are two four-inch discharge lines. one supplying standpipe in this building and one supplying standpipe in the adjoining LaSalle Extension University building.

The basement is used for building operations purposes only and houses hydraulic elevator machinery (including water tanks and pumps), oil-fired boilers, etc. The single stairway from the basement to the first floor is enclosed with a Class A fire door.

Corridor partitions on upper floors are plastered tile and pierced by ordinary glass doors with wood frames and movable glass transoms.

The interior finishes are mostly plaster. Dropped acoustic tile ceilings are found in the corridor and stairway only. The farthest corridor access door to the stairway or nearest elevator does not exceed fifty feet. It would be possible to adapt two outside elevator spaces for enclosed stairways.

The building connects with the LaSalle University Building to the north through openings in the basement, first, second and fourth floors. These openings are protected by sliding Class A automatic fire doors on both faces of walls. The wall between the buildings is about twenty-four inches wide (sixteen inch brick wall, bearing, on Manhattan Building; eight inch brick panel wall on LaSalle Building).

DEFICIENCIES: The principal deficiencies in this building, as compared to a new building constructed in conformance with today's building code, are the lack of enclosures around the single stairway and around the elevator shaft, the lack of a second stairway, and structural fire resistance below that required for a high rise building.

It appears to be desirable, from a restoration standpoint, to retain the original open grille work around the elevators.

RECOMMENDATIONS: For the Manhattan Building, the most effective alternative appears to be to equip the entire building with automatic sprinklers, and to arrange the ventilating system so as to pressurize the core in relation to the surrounding rooms. Another alternative is to construct enclosed stair-towers in the two end bays of the existing five bay elevator shaft.

REAL ESTATE REPORT

Despite the age of the Manhattan Building its elevator and stairway core is centrally located and compact. Also, the east-west and north-south symmetry is notable. As a result of this "modern" plan and the intelligent placement of critical shafts and stacks the net rentable area excluding the basement is approximately 119,000 square feet, giving a rentable-to-gross area ratio of roughly eighty-five percent (85%) which is deemed relatively efficient even by modern standards.

The motable visual element of the Manhattan Building is the variety and arrangement of the bay windows that project from the east and west facades. At the time the building was being designed two 12-story buildings were being constructed directly across Dearborn Street. Jenney decided to design bay windows to admit light to the lower floors that might otherwise have been blocked by the buildings across the street. This rationale is still valid today.

CURRENT USAGE: It was not possible to obtain accurate information as to the number and type of tenants in the Manhattan Building together with a small building lying between it and the Old Colony Building to the north, are owned and operated by LaSalle Extension University which is the major tenant in the Manhattan Building. The remaining tenants are principally warehouse and storage in character, along with a small office component. The ground floor space which at one time was used for retail purposes no longer serves a retail function although, if a demand were to arise, it appears well-adapted to that use and still retains much of its original structure and appearance.

MARKET ENVIRONMENT: As mentioned above, for purposes of exposition and adaptive reuse of the Manhattan Building is being examined which, for the time being, is inconsistent with known or anticipated market trends in the area. Currently there are no major residential structures within several blocks of the Manhattan Building but if the Printing House Row development becomes a reality there may be a substantial residential market in essentially loft space in part of the South Dearborn Street area between Congress Parkway and Polk Street. Nonetheless, at present there appears to be no justifiable reason for extensive adaptive reuse of the Manhattan Building to residential purposes.

SIGNIFICANT BUILDING CHARACTERISTICS: In the residential context the Manhattan Building does offer a number of interesting structural and design features that lend themselves to relatively simple adaptation to residential apartment or condominium uses. For instance, the symmetry of the building in plan is such that mirror images of the design for one-quarter of a floor could be made in the space. This symmetry leads to somewhat decreased cost of construction since standardization of components and key dimensions is possible and it contributes to the understandability of the apartment arangement for both tenants and visitors alike. The bay windows add a distinctly residential appearance to the east and west exposures and allow interesting sight lines up and down Dearborn Street and Plymouth Court.

The compact elevator and fire escape core can be adapted quite easily to comply with Chicago's stringent building code. On the matter of building code compliance it should be noted that whenever a building's usage is significantly altered, the new use must comply in all respects to the current codes.

STRUCTURAL REHABILITATION

Structural rehabilitation of old and historic buildings involves the conversion of interior space for alternative uses and the redesign of internal plumbing, heating, and electrical systems. Design alternatives often are limited by the need to retain the facade of a structure and other special or unique features of the former design. Outmoded or inefficient building systems must be replaced with newer, more efficient systems to minimize operational costs and problems; special attention must be given to energy costs. The concerns are best illustrated by the solution to the problem of rehabilitation of the landmark Manhattan Building in Chicago. Design plans called for conversion of the office space to apartments and for redesign of building systems. Design constraints included preservation of facade, interior grillwork, and marble. Project involvement spanned the initial survey of condition to project design. Appendix A provides detailed requirements for the project.

DESIGN PLANS

The rehabilitation of the Manhattan Project called for the conversion of offices to apartments. Ninety 1, 2, 3 bedroom rental apartments ranging in size from 900 to 2,500 square feet were planned. Floor plans for floors 2 through 10 and 11 through 16 were developed. These are presented in Figures 2 and 3. First floor commercial storefronts were retained, as well as the grillwork and marble on the upper floors.

PLUMBING SYSTEM

A brand new plumbing system was designed for each tenant space, along with individual meters for water heating costs. A new domestic water booster system was designed, using existing fire pump with modifications to meet the new duty and code revisions. Sprinkler systems were designed to meet codes for commercial space. The existing domestic water heaters in the basement were retained to serve the LaSalle extension school and the first floor commercial space. A new roof drainage system was designed for all three roofs.

HEATING, VENTILATING AND AIR CONDITIONING SYSTEM

Design plans called for each apartment to have a 3 to 5 ton heat pump with utility cost on tenant meter. The heat pumps would be fed from a central water system consisting of a colling tower for cooling and an electric boiler (to temper water) for heating. The apartment kitchens and toilets were exhausted by central system via roof mounted exhaust fans. The proposed design also called for interior corridors to be supplied with outdoor air which would be cooled in the summer and electrically heated in the winter. Utility rate differentials for electrical usage were planned for both tenants and the building. The existing gas fired steam boilers were retained to serve the (local) LaSalle extension school.

ELECTRICAL SYSTEM

Each apartment was designed with a 200 Amp electrical service. The electrical service to each apartment will be sufficient to accomodate the heat pump, the kitchen appliances, the domestic water heaters and a washer/dryer combination in addition to thenormal apartment receptacles and lighting. System design for the building service was 480V, 3 phase, 60Hz, while the apartment service was to be transformed from the building service to 240V, 1 phase, 60Hz.

IMPACT ASSESSMENT FOR REHABILITATION PROJECTS

The assessment of the impacts of rehabilitation projects involves the analysis of both site specific and areawide effects. An analysis of site specific impacts includes the review of factors such as the effect on the appearance of the structure, compatability of the converted or new use with adjacent uses, and transportation effects. Areawide impacts include effects of traffic generated by the project on areawide transportation systems, the effect of the project on development trends in the area (i.e., whether the project spurs additional development), and the contribution of the project to establishing a distinctive neighborhood character. Assessments of rehabilitation projects involve a multi-disciplinary tema of engineers, planners, and economists.

THE NEAR LOOP AREAWIDE ENVIRONMENTAL IMPACT STATEMENT (EIS)

The authors are involved in another project to assist the Near Loop Areawide EIS for the Department of Housing and Urban Development (HUD). This EIS involves the development and assessment of alternative environmental policies to guide HUD in the determination of whether financial assistance should be provided to proposed residential projects in the Near Loop Area. Development scenarios are being prepared and key issues are being identified. The redevelopment of the Near Loop Area is considered vital to the long term viability of Central Chicago.

A key issue identified by HUD for analysis in the EIS is the desirability of rehabilitation versus new construction. This issue is being researched to determine the potential for rehabilitation in the area, with several rehabilitation projects already underway. These include old printing a graphic arts buildings converted to apartments and condominiums; Warehouses and manufacturing structures renovated as loft apartments and studies, and so on. Analysis of this issue requires knowledge of state-of-the-art techniques for the economic rehabilitation of old structures and of the applicability of these techniques to specific sites in the Near Loop.

HISTORIC PRESERVATION

A related ussue is the potential effect on recognized historic structures. There are two Historic Districts in the Near Loop Area: The Prairie Avenue District and Printing House Row. Policies were developed to minimize effects of new projects on these important resources. The various groups, such as the State Historic Preservation Officer, the Chicago Landmarks Commission, and other public and private groups concerned with preservation during the assessment of these issues, were coordinated.

CONCLUSIONS

Rehabilitation of the Manhattan building as presented in this brief report demonstrates that old historic buildings can be preserved and renovated for uses different from the current occupancy to meet the present codes and criteria.

APPENDIX A

PROJECT REQUIREMENTS

PLUMBING: Each tenant space will have an all new plumbing system consisting of water closets, lavatories, tub with shower, kitchen sink with disposal, dishwasher, grease trap, washer, 50 gallon domestic electric water heater and piping. Utility costs for water heating will go on tenant meter.

The Building will be provided with a new domestic water booster system.

The existing fire pump will be up graded for the new duty and code revisions. There will be a fire protection standpipe in each stairway with a fire hose rack and protable fire extinguisher at each level. The commercial space on the first level will be completely sprinklered to satisify local codes.

The existing domestic water heaters in the basement will remain to serve the LaSalle extension school and the first floor commercial spaces.

New roof drainage system will be provided for all three roofs.

HEATING, VENTILATING AND AIR CONDITIONING: Each apartment will have a 3 to 5 ton heat pump with utility cost on tenant meter. The heat pumps will be fed from a central water system consisting of a colling tower for cooling and an electric boiler (to temper water) for heating. The utility cost for the tower, boiler and the pumping will go on the building meter.

The apartment kitchens and toilets will be exhausted by central systems via roof mounted exhaust fans.

The corridors will be supplied with outdoor air; this air will be cooled in the summer and electrically heated in the winter. The utility cost will go on the building meter.

Both tenants and building will have a utility rate differential for their electrical usage.

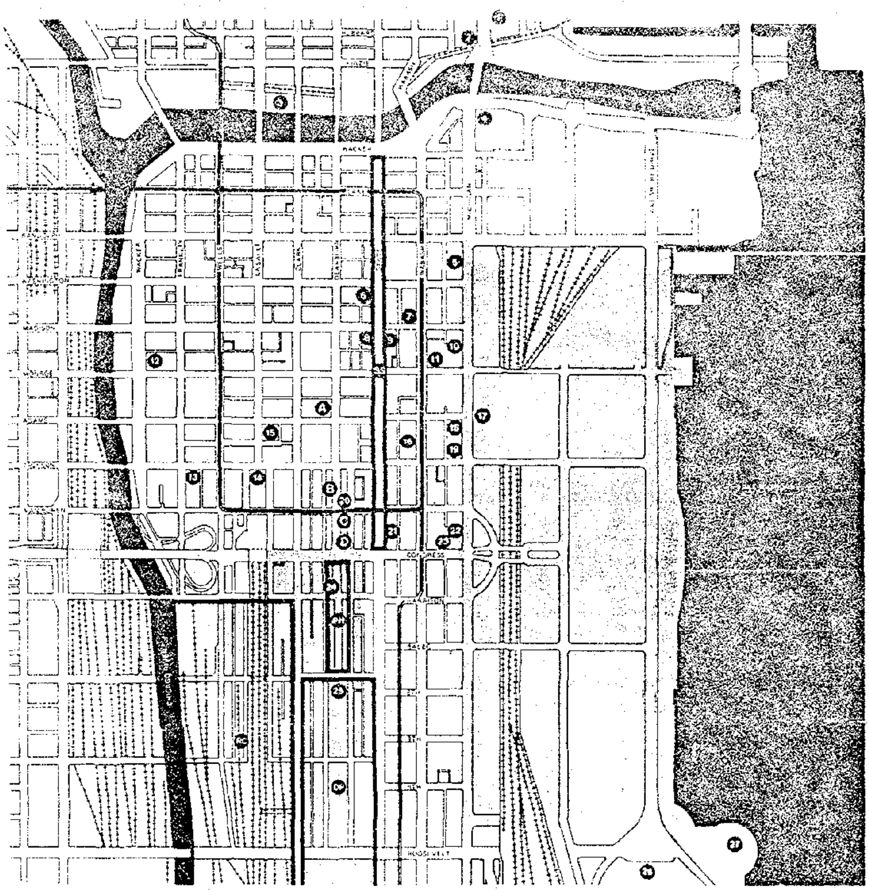
The existing gas fired steam boilers will remain to serve the LaSalle extension school.

Sufficient capacity is being provided in the central system to accomodate the first floor commercial spaces.

ELECTRICAL: Each apartment will be provided with a 200 amp electrical service. The meters will be banked in the electrical equipment rooms and the circuit breaker panels will be located within the apartments. The electrical service to each apartment will be sufficient to accomodate the heat pump, the kitchen applicanees, the domestic water heaters and the washer/dryer combination in addition to the normal apartment receptacles and lighting.

The building service will be 480V, 3 phase, 60Hz; the apartment service will be transformed from the building service to 240V, 1 phase, 60Hz.

FIGURE 1



LANDMARK DISTRICTS

LANDMARK DISTRICTS

FOUR LANDMARK BUILDINGS

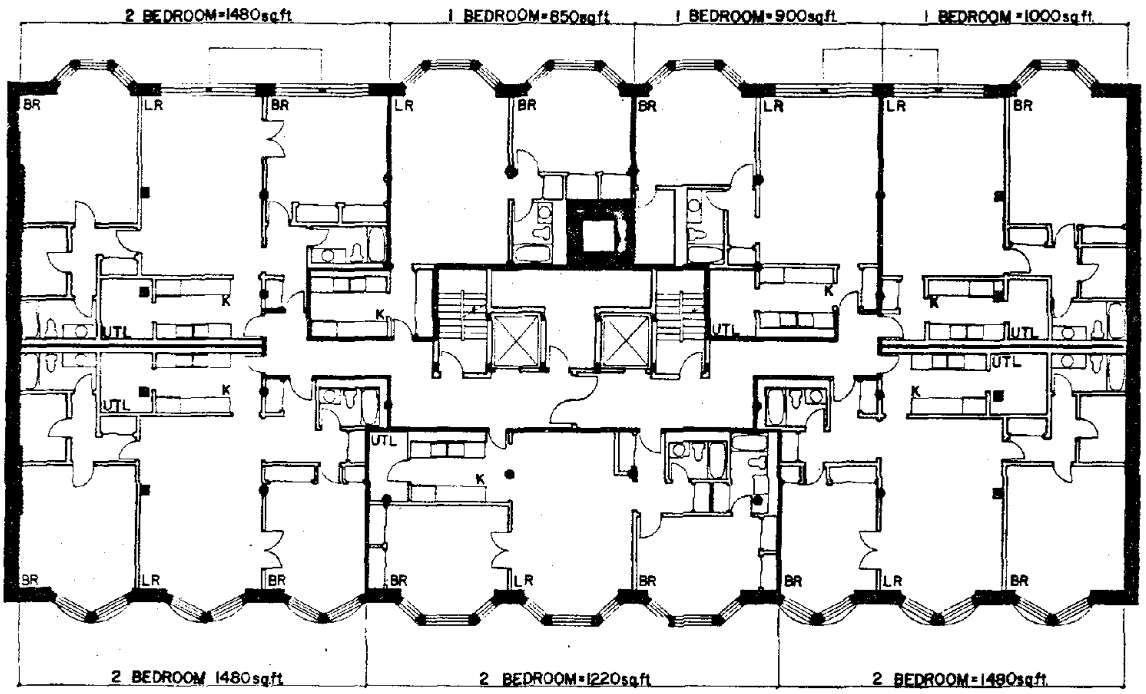
A MARQUETTE BUILDING
B MONADNOCK BUILDING
C OLD COLONY BUILDING
D MANHATTAN BUILDING

LANDMARKS

1 TRIBUNE TOWER
2 WRIGLEY BUILDING
3 REID MURDOCH BUILDING
4 333 NORTH MICHIGAN AVENUE BUILDING
5 CHICAGO PUBLIC LIBRARY
6 RELIANCE BUILDING
7 WIEBOLDT'S WABASH AVENUE SECTION
8 CHICAGO BUILDING
9 CARSON, PIRIE, SCOTT & COMPANY
10 GAGE GROUP
11 CHAMPLAIN BUILDING
12 TROESCHER BUILDING
13 BROOKS BUILDING
14 BOARD OF TRADE
15 THE ROOKERY
16 PAKULA BUILDING
17 ART INSTITUTE OF CHICAGO
18 CHAPLIN & GORE BUILDING
19 RAILWAY EXCHANGE
20 FISHER BUILDING
21 SEARS, ROEBUCK & COMPANY
22 FINE ARTS BUILDING
23 AUDITORIUM BUILDING
24 PONTIAC BUILDING
25 POLK STREET STATION
26 FIELD MUSEUM OF NATURAL HISTORY
27 SHEDD AQUARIUM

DEVELOPMENT PROJECTS

PH PRINTING HOUSE ROW (PROPOSED)
RC RIVER CITY (PROPOSED)
DP DEARBORN PARK
SS STATE STREET MALL

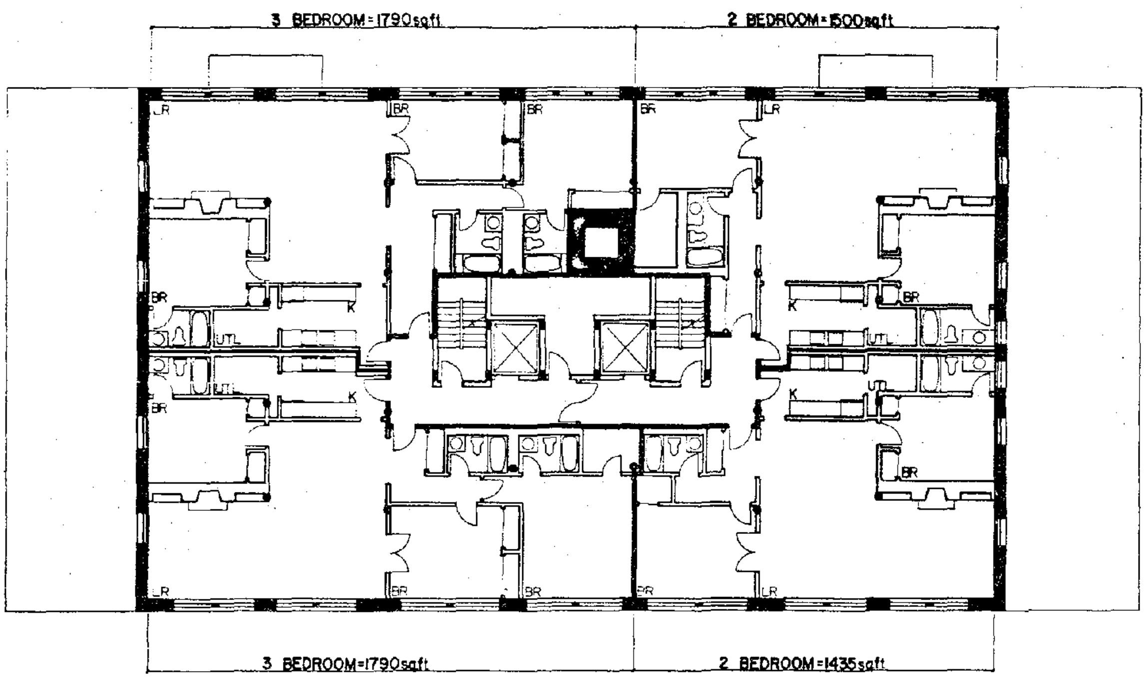


DEARBORN STREET

0 5 10 20 30

FIGURE 2

FLOOR PLAN FLOORS 2 THRU 10



DEARBORN STREET

0 5 10 20 30

FIGURE 3

FLOOR PLAN FLOORS 11 THRU 16

REPAIR AND REHABILITATION OF CONCRETE LOCK WALLS

Tony C. Liu
Office of Chief of Engineers
Department of the Army
Washington, DC 20314

SUMMARY

This paper presents the major work involved in the repair and rehabilitation of one of the lock structures on the Monongahela River near Pittsburgh, Pennsylvania. Particular emphasis is placed on the techniques and methods used for condition evaluation, concrete removal, dowel installation, concrete refacing, and shotcrete repair. The evaluation of concrete performance in the repaired lock walls is also presented.

INTRODUCTION

The U. S. Army Corps of Engineers operates and maintains over 200 concrete navigation locks in the navigable rivers of the United States. The function of these navigation locks is to create the stepped succession of navigation pools within a waterway. Of the total number of lock structures, more than half are over 50 years old. With structures of this advancing age, major repair and rehabilitation is often required to assure the safety and continuation of the locking capacities of the structures. This paper presents the methods used in the repair and rehabilitation of Locks and Dam 3, Monongahela River.

Locks and Dam 3 is located on the Monongahela River in Allegheny County immediately upstream of the town of Elizabeth, Pennsylvania. It is about 24 miles (38.6 Km) upstream from Pittsburgh, Pennsylvania. The structure consists of one 56 ft by 720 ft (17 m by 219.5 m) and one 50 ft by 360 ft (17 m by 109.7 m) lock chamber with a non-navigable fixed crest dam spanning 688 ft (209.7 m) from the river face of the river wall to the river face of the abutment. All the walls, except the upper guard wall extension, are concrete gravity type founded on rock. The original locks and dam were constructed from 1905 to 1907, and have been operational since May 1907. It became apparent in the early 1970's that major repair would be required to assure the continuation of the locking capability of the project. The condition of the concrete was in an advanced stage of deterioration. The operating machinery was obsolete and required frequent and costly repairs. As a result, a major rehabilitation plan was formulated and carried out which included refacing lock walls, anchoring unstable walls, replacing the upper guard wall extension, renovating gate and valve operating machinery, and renewing the electrical system. Work began in July 1978 and was completed in December 1980.

In this paper, the techniques and methods used for repairing and refacing the concrete lock walls will be presented. Other work items such as renovation of the mechanical and electrical systems will not be covered.

CONCRETE CONDITIONS

An engineering condition survey and structural investigation was conducted during the period from October 1974 through June 1975. The objective of this study was to conduct a detailed condition survey of the structure along with an in-depth engineering evaluation of the stability and stress analyses of the lock monoliths. The work included measurements for cracking, alignment, settlement, and surface quality of concrete, foundation exploration, materials testing, and structural stability and stress analyses. The results of this investigation showed that the lock walls were badly spalled and scoured (Figure 1).

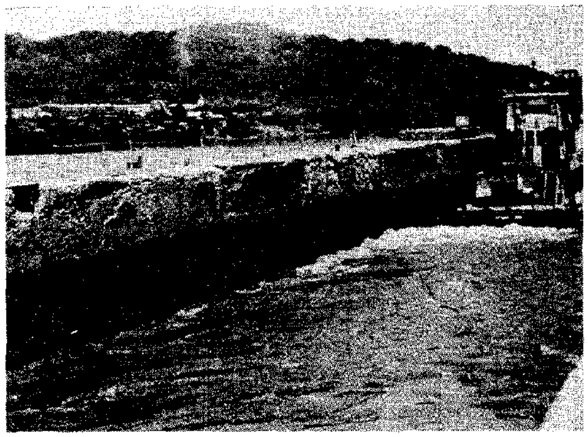


Figure 1. Deteriorated lock chamber.

The worst conditions were between pool levels and at monolith joints. The concrete cores and the borehole photograph showed that the top 2 to 6 feet (0.6 to 1.8 m) of concrete were badly deteriorated. The cores contained fairly close spaced parallel fractures. Major cracks existed in gate monoliths. The crack in one of the gate monoliths was so extensive that a large block of concrete was isolated from the rest of the monolith.

Freezing and thawing action, acid water attack as a result of acid mine drainage, boat impact and abrasion on poor quality non-air-entrained concrete all contributed to the deterioration of this old structure.

LOCK WALL REPAIRS

To prevent further deterioration and failure of the concrete lock walls, extensive rehabilitation work was necessary. In some areas, this was accomplished by removing a specified thickness of deteriorated concrete and replacing it with new reinforced concrete facing. In other areas, a new surface was placed over the old using shotcrete.

Concrete Removal

Approximately 12 in. (0.3 m) of concrete were removed from the chamber faces of the lock walls. Blasting was chosen as the most efficient method of removal. Prior to initiation of the work, test blasting was conducted on the river wall to determine the optimum parameters. Two blasts were detonated in the tests. All holes were 2 in. (51 mm) in diameter, 19 ft (5.8 m) deep, and 1 ft (0.3 m) from the face of the wall. Each hole was loaded using two or more lengths of 50-

grains-per-foot (10.6 grams-per-metre) Primacord taped along opposite sides of a 1-1/2 in (38 mm) diameter wood pole. Stemming in the collar was wet sand. Four transducers were located in the vicinity of the blasting area to record vibration intensity. Transducer no. 1 was located on the test monolith, directly behind the blast and 1 ft (0.3 m) from the outside face of the wall; No. 2 was on the adjacent downstream monolith; No. 3 was on the adjacent upstream monolith; and No. 4 was on the middle wall monolith directly across from the test monolith. Test data are summarized in Table 1. (McElfresh and Remaly, 1981)

Table 1. Test Blast Data

Blast No.	No. of Holes	Explosive grains/ft (Grams/m)	Spacing in. (m)	Transducer Location	Particle Velocity ft/sec (m/s)
1	4	200 (42.5)	12 (0.3)	1	4.3 (1.3)
	3	150 (31.9)	12 (0.3)	2	7.2 (2.2)
				3	4.0 (1.2)
				4	0.4 (0.1)
2	13	100 (21.3)	6 0.2)	1	6.6 (2.0)
				2	2.8 (0.9)
				3	4.9 (1.5)
				4	0.2 (0.1)

These tests each produced clean breaks with no damage to the remaining structure. From these results, a decision was made to try 100-grains-per-foot (21.3 gram/m) at 12-in. (0.3 m) spacing for production blasting. Blasting for removal of concrete within the chambers was done with water at lower pool level to help dampen the effects of the blasting. The selected charge of 100-grains-per-foot (21.3 gram/m) worked well above the waterline but produced inconsistent results below the waterline. To compensate for the cushioning effect of the water, a charge of 200-grains-per-foot (42.5 grams/m) was used below the waterline and the desired results were achieved.

Installation of Dowels

After the deteriorated concrete was removed from the face of the wall, 1.5 in (38 mm) diameter holes were drilled on a 10 degree inclined angle on 2-ft (0.6 m) centers for No. 6 hooked dowels. (Figure 2)

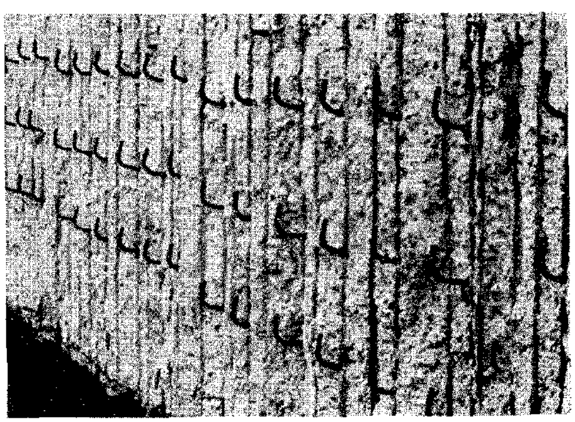


Figure 2. Hooked dowels installed in river face of middle wall

The dowels were bonded in the drilled holes using polyester-resin cartridges. A cartridge was inserted into each hole and the dowel was installed and spun with an air drill. All dowels had a miminum embedment of 15 in. (380 mm). A field pullout test was conducted on selected dowels to evaluate the pullout performance of dowels installed under field conditions. The results indicated that the performance of dowels installed under field conditions using the polyester-resin cartridges was acceptable. None of the dowels failed to hold at least the design stress of 40,000 psi (276 M pa)
A recent study on the performance of dowels used in anchoring concrete facing to vertical lock walls indicated that the dowel did not significantly increase the load carrying capacity of the test specimens (Liu and Holland, 1981). The quality and integrity of the bond along the interface between the new and existing concrete was a much more significant load transfer mechanism than the amount of dowel reinforcement present. The test data indicated that No. 6 dowels at 4-ft (1.2 -m) spacing are adequate to insure that a ductile failure mode would prevail.

After the hooked dowels were installed, a vertical reinforcing mat of No. 5 bars on 12-in. (0.3-m) centers was then positioned approximately 4 in. (0.1 m) from the eventual face of the wall.

Concrete Refacing

The concrete mixture used for lock wall refacing had a water-cement ratio of 0.49 by weight and 7% air entrainment. Crushed limestone coarse aggregate (1.5 in (38 mm) maximum size), natural sand, and ASTM Type II portland cement were used. The concrete was mixed in a floating batch plant which had two 4 cu. yd (3.1 cu. m) mixers. The use of a floating batch plant minimized the time between mixing and placing, and eliminated the need to handle the concrete more than once. The floating batch plant contained automatic scales for batching. A sensor in the sand bin monitored the moisture content and the batch water was automatically adjusted. A derrick boat was used to move the concrete buckets directly from the floating batch plant to concrete form.

The concrete was consolidated by external vibrators. Immediately after form removal (approximately 2 days after concrete placement), the concrete surfaces were sprayed with concrete curing compound. The partially completed lock wall refacing is shown in Figure 3.

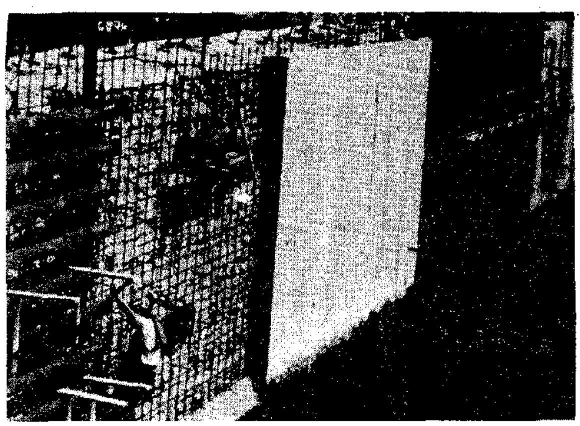


Figure 3. Partially completed lock wall.

Shotcrete Repair

The river side of the river wall, the lower guide wall, all existing gate recesses and various areas on the upper guide wall and middle wall were treated with shotcrete. The wall faces were prepared by removing the deteriorated and loose concrete with air chipping tools and a bush hammer head mounted on a hoe ram. Some wall surfaces required only sandblasting. A high-pressure water jet was then used to clean and wet the surface. Shotcreting was done using the dry-mix method with water added at the nozzle. The sand/cement ratio was approximately 4.0.

Evaluation

An inspection of the lock walls was made in May 1981, approximately 6 months after the completion of the rehabilitation work (Figure 4).

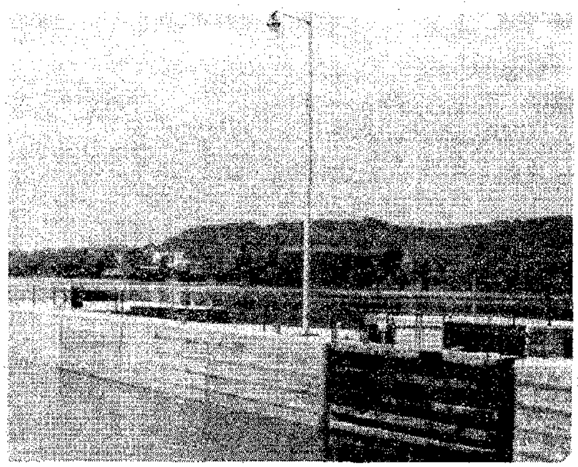


Figure 4. Refaced river wall.

The new concrete facing on river wall and middle wall was generally in good condition with only minor cracking. However, several vertical and horizontal cracks were noted on the new concrete facing on land wall. It was believed that the cracking was mostly the results of excessive thermal and shrinkage stresses which were induced by the severe restraint of the existing concrete. The performance of the shotcrete coatings was generally not good. Several areas of the new shotcrete were abraded and spalled by tows entering and leaving the locks. The shotcrete on the river side of the river wall (area without tow impact) was in good conditions with only minor shrinkage cracking.

CONCLUSIONS AND RECOMMENDATIONS

(1) The use of blasting to remove concrete from the lock wall faces proved to be a very efficient method. This allows a contractor to do the major part of the removal work (i.e., line drilling) without closing the lock chamber.

(2) The polyester-resin cartridges were an effective bonding agent for anchoring dowels in concrete, provided that adequate embedment lengths have been obtained and that the cartridge manufacturer's installation recommendations have been followed.

(3) The No. 6 dowels at 4 ft (1.2 m) spacing are adequate for anchoring replacement concrete to existing lock walls.

(4) Thin shotcrete is not suitable for refacing lock walls where abrasion and impact by tows are expected.

(5) The severe restraint provided by the existing concrete is an unique problem in lock wall refacing jobs. This problem deserves a R&D study which should include determination of the maximum acceptable temperature differental between ambient and concrete, optimum mixture proportions, proper construction procedures, and maximum permissible crack width. The results of this study would be useful for future lock wall refacing work.

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PERFORMANCE STUDIES OF GRID DECKS FOR OFFICE FLOORS

Hota V. S. Gangarao,
Professor Civil Engineering Department
West Virginia University
Morgantown, West Virginia 26506
U. S. A.

SUMMARY

This report summarizes the manufacturing and erection practices and highlights the maintenance problems of grid decks. Causes of failures of main and secondary bars of open grid decks and maintenance problems of concrete filled decks are established and corrective measures are suggested. The present AASHTO design philosophies and other criteria for transverse load distribution and stiffness are critically reviewed. Stress improvement factors and rating improvements of bridges utilizing open or filled grid decks in place of concrete decks are investigated. Certain design manufacturing and erection alternatives are recommended, to maximize the grid span between stringers and to minimize maintenance problems. Specific recommendations are made for future research in this area.

INTRODUCTION

Undertaking a bridge replacement program in the United States of America involves large amounts of money. Even though the Federal Government is spending over $1 billion (Rs. 300 crores) annually, the estimated cost of proper replacement of deficient bridges is around $10 billion (16)*. Under such severe fiscal constraints, the most economical approach to improve the safety of bridges is to investigate the existing rehabilitation techniques and suggest cost-effective ways of restoring integrity of old bridge structures. One of the rehabilitation techniques is to redeck a bridge with open- or concrete-filled steel grid panels.

In general, the steel grid panels (Figure 1) have been successfully used as bridge decks with the idea of providing a speedy and economical solution for rehabilitation purposes (18) and also as "more permanent" decks on new bridges. This can be attributed to their highly favorable improvement factor (change in flexural requirements between the base system and the one employing dead load reduction technique) based on cost per square yard, when compared with the improvement factors of other bridge superstructural systems (6).

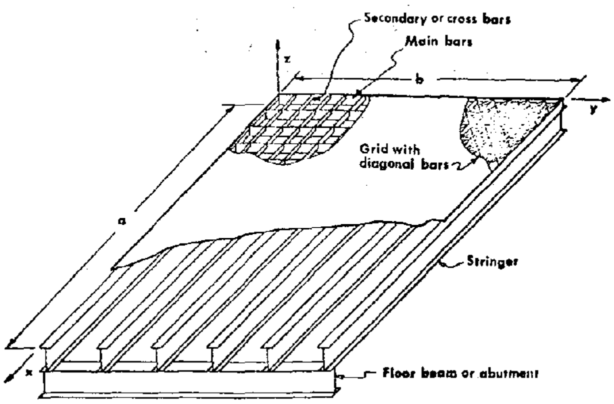

Figure 1 Open Grid Deck/Wide Flange Stringer Bridge System

* Additional details are in the list of References.

Steel grid floors may be broadly classified into two categories: (a) open grid deck system; and (b) concrete-filled grid deck system. Concrete-filled systems may be further classified as fully-, partially-, or overly-filled systems.

While the open grid decks have found application in the rehabilitation and construction of short span, medium span and movable bridges, the concrete-filled decks have been widely used for all types of bridges, including many major bridges such as the Walt Whitman Bridge (1956); Mackinac Straits Bridge (1957); Verrazano Narrows Bridge (1964); and so on.

In spite of the extensive use of steel grids as bridge floors over the past fifty years in the United States, there is little understanding of their behavior and performance due to the increased use of deicing chemicals, excessive allowance of differential elevation in stringers, improper functioning of rocker/roller supports, and ever increasing intensities of (wheel) loads. This lack of understanding is leading to improper designing and detailing, and serious maintenance problems.

The overall objective of this research is to arrive at steel grid deck systems that would carry legal loads with minimal maintenance problems over a fifteen- to twenty-year period, and to utilize these systems to rehabilitate the decking of old bridges with the idea of improving the bridge ratings.

A three-phase research program is designed to deal with the short- and long-range performance studies, and design manufacturing and erection aspects. The three phases are: (a) feasibility study; (b) laboratory and field testing to check the results obtained from the feasibility study; and (c) accelerated studies related to long-range maintenance and software development for rehabilitation purposes.

More specifically, objectives of this paper are:

(a) Take an inventory of bridges in this region to recommend a percentage of extant bridges that can be rehabilitated with grid panels;

(b) Compile and synthesize data related to fabrication, erection and serviceability of several grid types;

(c) Critically review the present design philosophies given in AASHTO (1); and

(d) Recommend certain modifications in size and manufacturing process to minimize maintenance problems.

Inventory of Bridges

This portion of the work is concerned with the compilation and synthesis of data related to fabrication, erection and serviceability of several grid types. Also, inventory of bridges in this region and in a few other states is undertaken to establish a significant data base that can be utilized in checking the validity of design methodologies and equations that are proposed herein. Based on the information obtained from this inventory, a percentage of bridges that can be rehabilitated with grid decks is recommended.

The general classification of this inventory includes bridge types, span lengths, widths, etc. Additional information related to grid types, fabrication details, traffic count, accident data and failure types are solicited through a survey.

Compilation and Synthesis of Data

Even though many fabrications and field observations related to manufacturing, transportation, erection and maintenance have been made, information on erection problems, grid panel distress and maintenance problems are not available in the form of published literature. However, it has been well established from the questionnaire that field data on visible distress are available in the form of raw data sets with several state highway agencies and the manufacturers of grid panels. These data sets are collected through questionnaires, telephone conversations and also through personal visits to several bridge sites where deck distress problems are known to be severe.

These data sets are tabulated in Reference 8 and are synthesized and summarized in the following section. During the synthesis of the problem, special emphasis is given to deck failures (failure of main and secondary bars in open grids, buckling of deck or closing of joints in concrete-filled grids), weld failures between the grid deck and stringers, effects of differential elevation of stringers on grid deck stresses (installation), traffic count, skid resistance and rider annoyance, accident data, and maintenance problems.

During the field visits, the need and importance of the use of grid deck bridges and their performance problems on low-volume as well as medium- to high-volume traffic roads have been carefully studied.

Review of Design Philosophies

The historic development of the design guideline as given in the present specifications (1) for transverse load distribution, size and spacing of main and secondary bars, and size and length of weld between the grid panels and stringers is reviewed and assessed in terms of traffic intensity, allowable stresses and structural distress.

PERFORMANCE OF GRID DECKS

This section synthesizes and summarizes the information on steel grid decks that has been compiled through questionnaires, field trips and telephone conversations. This information includes the fabrication and erection details of several grid types. Also, it includes the long- and short-term performance aspects, and other data such as traffic count and accident rate.

Inventory of bridges in this region and in a few other states is undertaken to establish the following data base: bridge types and their geometric details such as span length, width and skew angle; grid types including fabrication details, erection procedures, traffic count and maximum load intensity; accident data relating to decks, types of deck failures and other problems such as skid resistance, rider annoyance, and so on. This inventory is conducted primarily through a questionnaire.

Open Grid Decks

Open steel grid panel flooring is available in a variety of configurations from several manufacturers. The steel grid panels functioning as bridge decks are usually welded to the top flange of wide flange stringers or floor beams which transmit truck loads to abutments. The elevation changes (along the bridge widths) in rockers supporting stringers and intermediary shims between stringers and grid panels are necessary to form the roadway crown.

The open grid is very light, i.e., only weighs 15 lbs. per square foot of a deck. It has the advantage of washing down the deicing chemicals without providing any special bridge drains. However, it is found that the open decks are somewhat noisy, slippery when wet, and lead to excessive rusting of supporting floor beams or stringers. Hence, construction details should be developed to eliminate trappings (of debris and chemicals) over main stringers.

Maintenance Problems

Based on field observations and the national survey conducted as a part of this research, the following failures have been commonly observed and the causes of such failures are noted.

(a) As shown in Figure 2, twisting and fracture of cross bars and diagonals (if any) near the joints lead to progressive failure of an open deck. This is due to inadequate shear resistance of secondary bars, and the detailed proof is given in Reference 8.

(b) Failure of plug welds connecting main and secondary bars makes the grid lose its integrity. This is attributed to excessive amplitudes of vibration caused by moving trucks and to the lack of adequate contact area of the plug welds between the main and secondary bars. Usually such plug welds are provided to keep the bars together. However, they try to transmit forces from one bar to the other and fail due to inadequate resistance.

(c) Failure of weld between the main bars of an open deck and the steel stringers. This may happen due to poor quality field welding. Also, this is related to the inadequacy of weld sizes (depth and length) and the lack of proper resistance to transfer the horizontal shear forces (developed through composite action between the deck and the stringer) from the grid panels to stringers.

(d) From field observations, failure of main or secondary bars is generally located between stringers and under the "wheel tracks" where the vehicle takes a natural course of adjustment. This type of failure usually takes place because of inadequate resistance of cross bars to braking, or accelerating forces. Such failures are found to be common in bridges located on a grade or on steep horizontal curves or in situations where the vehicles have to take 90° turns, developing huge in-plane centrifugal forces that cannot be resisted by secondary bars. Also, this may be due to the lack of proper splicing of joints between two consecutive grid panels, which may have varying vertical elevations, thus creating "bump" effects. Finally, lack of guidelines on the maximum clear distance between two adjacent stringers, leading to excessive grid panel deflections, is contributing to serious grid panel member failures at an accelerated pace.

(e) Possible fatigue failure of welds connecting the main and secondary bars, or of welds connecting the grid panels and stringers.

(f) Failure of cross bars due to accidental interlocking of foreign objects between the openings, which may be subjected to dragging by vehicles.

(g) Skid resistance of certain types of grids is found to be poor (skid resistance number is in low 30s), whereas the skid resistance number with certain other types ranged between 60 and 70. This depends, obviously, upon the groove size and spacing on the top of the main and secondary bars. Hence, a designer has to be careful in recommending a type of an open deck panel that would result in the skid resistance number of at least 50. Additional or more specific guidelines have to be developed before making any specific recommendations.

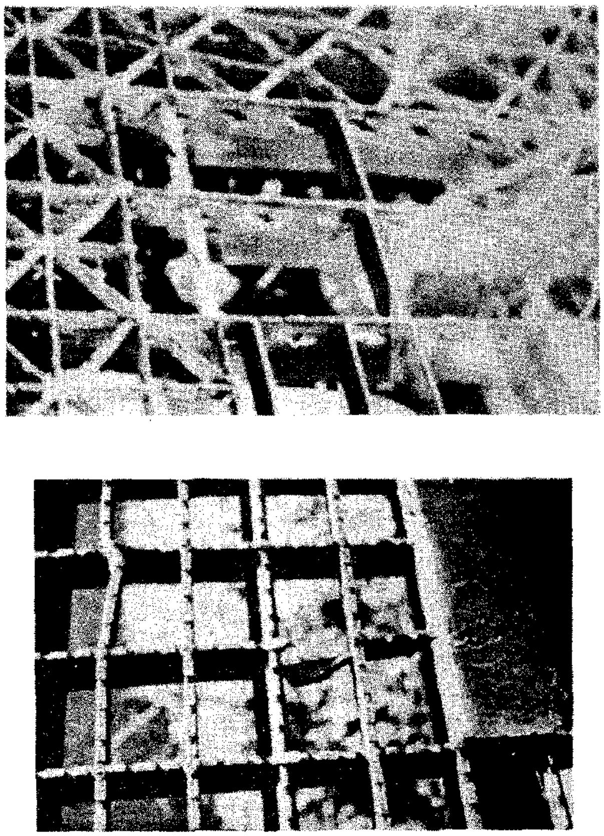

Figure 2. Failure Types and Corrective Measures of Open Grids

Summary of the Open Grid Deck

The open steel grids are found to be very economical systems either for rehabilitation purposes or for temporary as well as permanent decking. However, they are exhibiting severe maintenance problems. The types of problems and their causes have been identified from field observations, telephone conversations and information collected through the national survey. Our studies clearly indicated that the majority of failures can be traced to inadequate resistance of details (8), i.e., insufficient understanding of grid panel components and joints, rather than the understanding of global behavior of grid decks.

Some difficulties in proper fitting of grid panels in the field have been reported. This is commonly overcome by field modifications. Special precautions have to be taken in preventing the welding of grid decks to stringers (with differential elevations in excess of 1/2") by externally forcing the grids to be in contact with the stringer. Differential elevations in excess of 1/2" have to be corrected through shimming. Better welding procedures have to be adopted to achieve reliable design strengths under severe loading conditions. It appears that the serrations have to be minimized to minimize the stress concentration effects on local members without appreciably decreasing the skid resistance of the open deck. Tentative recommendations in terms of the size and spacing of grid components and weld sizes are recommended in the latter part of this paper to achieve a more durable open deck.

Concrete-Filled Grid Decks

Concrete-filled steel grid decks are commonly of three types: (a) partially-filled (half the depth of the main bars); (b) fully-filled decks; and (3) overly-filled (1/2" to 1" above the top of the main bars).

It has been reported that filled grid decks are generally performing to the satisfaction of their clients even after thirty-five to forty years of servicing and salting. Even though certain performance problems have been reported (10, 12, 19), some bridge engineers reported (3, 14) that filled decks have higher levels of resistance to wear, corrosion and weathering. Their higher resistance to corrosion is attributed to the physical arrangement of concrete and steel in filled decks and good grounding of the deck which minimizes formation of stray currents responsible for local corrosion problems.

Concrete-filled grids certainly improve skid resistance and eliminate the maintenance problems of open grids; however, the additional weight of concrete reduces the live load capacity of the bridge and requires an adequate deck drainage system.

Maintenance Problems

Even though concrete-filled grid decks seem to be performing very satisfactorily over long periods of time, they are by no means free of maintenance problems. Some of these problems are described below (15, 19, 21):

(a) Longitudinal growth of concrete-filled grids: This has been observed from broken welds between the grid deck and main stringers and "popping out" of concrete from the grid deck cells. In addition, humping of the filled deck, breaking of curtain walls of abutments and closing of expansion joints have been observed. The reasons for such growth are two-fold: (1) growth in the weaker (lower percent of steel area, i.e., along cross bars) direction of the filled deck due to corrosion and rusting, and consequent expansion of the deck; this seems to have been minimized by providing waterproofing membranes on the decks; (2) longitudinal deck growth due to cracking of grid deck at the joints between adjacent grid panels (usually 6' or 8' intervals), where adequate welding has not been provided. Proper splicing of cross bars and main bars seems to be preventing such problems. Bridges experiencing longitudinal deck growth created "bump" effects near expansion joints and/or approach slabs, and the cracking of deck led to severe corrosion of stringers (Figure 3).

(b) Cracking of 1/2" to 3/4" thick concrete as a cover or overfill: Either the deck should be covered by asphaltic wearing surface of at least 2-1/2" thickness or the top surface or grid bars should be left exposed. Concrete cover of 1/2" to 3/4" thickness leads to cracking of the overlay and trapping of salts on decks (Figure 3).

(c) Slipperiness of the deck: This can be prevented by providing a blacktop or wearing course of 2-1/2" thickness; however, such wearing surfaces are leading to secondary problems, i.e., creating temperature differential along the deck depth and inducing additional longitudinal movements and stresses.

(d) Failure of welds between stringers and filled decks: This can be prevented by providing adequate high-quality welding to resist horizontal shear between the main stringers and filled deck. However, if the primary failure is due to longitudinal expansion of the deck, weld failures can be minimized by preventing the longitudinal deck growth.

(e) Cupping: This is a natural wear which is directly proportional to the deck longevity and the average daily traffic (ADT) on the bridge. This problem can be avoided by periodically applying the concrete as the wearing surface.

Summary of the Filled Grid Deck Survey

Even though concrete-filled grid deck performance is generally found to be far superior to cast-in-place concrete or open grid decking, some of the recent field observations by the author revealed that the longitudinal growth of these decks is turning out to be a more serious maintenance problem than has been reported in the literature. Other performance problems such as failure or welds between main stringers and the filled deck, cracking of 1/2" cover and cupping of roadway have been observed, also.

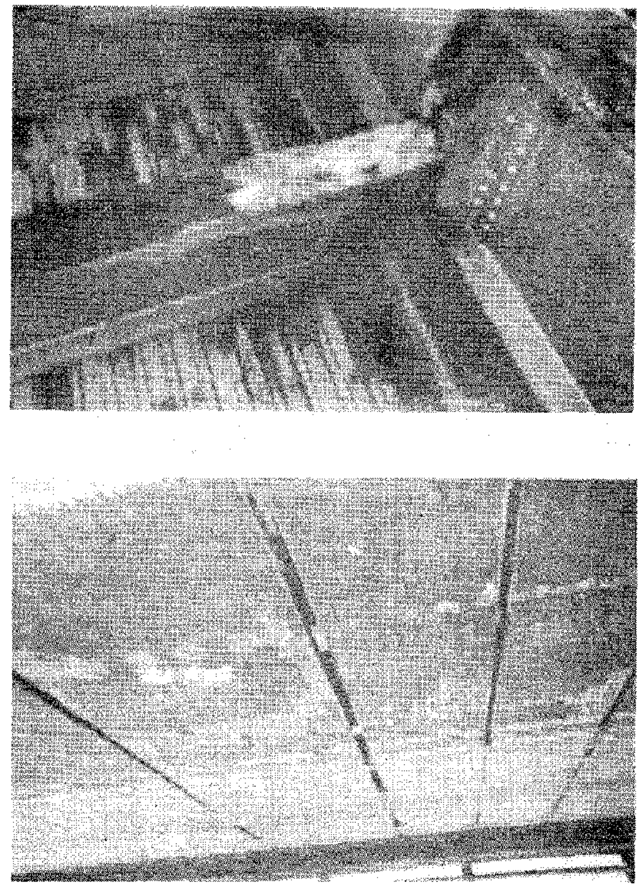

Figure 3. Concrete-Filled Grid Deck Distress

To prevent or minimize the longitudinal growth of grid decks, the following precautionary measures are strongly recommended:

(a) Joints between adjacent grid panels be properly spliced by welding all cross bars and rigidly connecting the main bars to stringers.

(b) Fill the grid deck with concrete having corrosion retardant admixtures (calcium nitrite) and provide waterproofing membrane to prevent the deicing chemical from penetrating into the concrete cells of the deck.

DESIGN PHILOSOPHIES

The responses of our survey revealed that all of the state highway agencies, with the exception of Pennsylvania Department of Transportation of U.S.A., are totally relying on the open steel grid deck transverse load distribution formula suggested by the current AASHTO Standard Specifications for Highway Bridges (Article 1.3.6(C)), which is "A wheel load shall be distributed normal to the main bars, over a width equal to 1-1/4" per ton of axle load plus twice the distance center of main bars. The portion of the load assigned to each main bar shall be applied to the bar uniformly over a length equal to the rear tire width (20 inches for H20, 15 inches for H15)".

or $x = 1.25T + 2S$

where x = width of the area over which load is distributed
T = axle load in tons
S = spacing of main bars

Normally, one would expect an increase in the load intensity as the spacing of main bars increased. However, as reported at first by Hasija (19), the AASHTO distributions formula led to a decrease in load intensity with increasing bar spacings (refer to Table 1), which is erroneous. In other words, for open grid floors it is not the area of the floor that takes the pressure, but a given number of main bars per unit area have to transmit the tire pressure on the stringers.

Unfortunately, AASHTO does not have any specific guidelines for transverse strength of stiffness. Article 1.3.6(A) of AASHTO Specifications states that the strength and details of transverse steel shall meet with the approval of the engineer. The engineering judgment has varied considerably in this regard because of the absence of systematic testing and other detailed stress analysis. A few simple design calculations are given in Table 1 to prove the abovementioned deficiencies, related to failures. A comprehensive analysis of grid decks was performed by the author (8) and Ball (5) to avoid the inconsistencies in design. The results are not presented herein, however, due to space limitations.

Spacing	AASHTO Distribution x (inches)			Intensity of Pressure Over the Contact Area (psi)		
(inches) S	16T	12T	8T	$\frac{16000}{20(x)}$	$\frac{12000}{15(x)}$	$\frac{8000}{10(x)}$
1/4	20.5	15.5	10.5	39.02	51.51	76.19
1/2	21	16	11	38.10	50.00	72.73
1	22	17	12	36.36	47.06	66.67
2	24	19	14	33.33	42.11	57.14
4	28	23	18	28.57	34.78	44.44
6	32	27	22	25.00	29.63	36.36
8	36	31	26	22.22	25.81	30.77
12	44	39	34	18.18	20.51	23.53

Table 1. Intensity of Pressure Based on AASHTO Load Distribution (15)

Cross Bar Stresses of Open Grid Decks under Longitudinal Forces

Article 1.2.13 of AASHTO Specifications on Longitudinal (braking) Forces states, ". . .provision shall be made for the effect of a longitudinal force of five percent of the live load in all lanes carrying traffic headed in the same direction". Herein, 10% of wheel load is considered for finding longitudinal forces. This includes an impact effect of 100% of the longitudinal force, which is .05 of wheel load according to AASHTO Specifications. It should be noted that the proposed value is much less than the suggestions. The Canadian studies have led to upward changes in determining longitudinal forces that are as high as 80% of the wheel load (21). A general distribution of longitudinal forces on a typical open grid deck is shown in Figure 4, and it is properly accounted for determining the induced bending and shear stresses of cross bars.

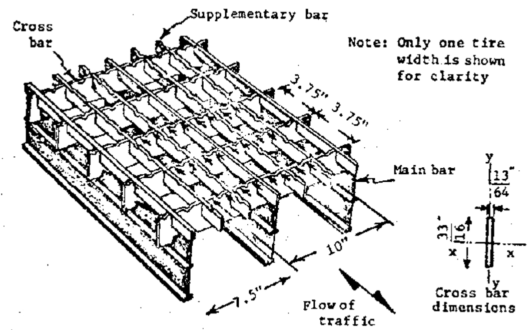

Figure 4. Longitudinal (braking) Forces on Cross Bars

$$\text{Longitudinal force/inch of cross bar} = (0.1)(16)\frac{3.75''}{20''\times 10''} = 0.03 \text{ k/in.}$$

where 16^k is the load transferred from dual tires of a HS20 truck.

The contact width and length of dual tires are taken as 20 and 10 inches, respectively. The suggested dimensions are slightly modified values of AISC Specifications for the design of orthotropic steel plate deck bridges. Additional laboratory tests on grid decks (9) also revealed that the 10" x 20" contact area was reasonable towards calculating bending and shear stresses of cross bars. Even though the cross bar dimensions are about 33/16" x 13/64", for calculation purposes, the dimensions of cross bars, herein, are assumed to be 33/32" x 13/64" because of notching to accomodate the cross bars.

Bending stress due to longitudinal forces at y-y axis:

$$= f = \frac{MC}{I} = \left(\frac{0.03 \times 7.5^2}{8}\right)\frac{\left(\frac{1}{2}\times\frac{13}{64}\right)}{\frac{1}{12}\left(\frac{33}{32}\right)\left(\frac{13}{64}\right)^3} = 29.8 \text{ ksi}$$

Shear stress due to longitudinal forces at y-y axis:

$$= \frac{VA\bar{y}}{It} = \frac{\frac{(0.03 \times 7.5)}{2}\overbrace{\left(\frac{33}{32}\right)\left(\frac{1}{2}\times\frac{13}{64}\right)}^{A}\overbrace{\left(\frac{1}{2}\times\frac{1}{2}\times\frac{13}{64}\right)}^{\bar{y}}}{\underset{b}{\left(\frac{1}{12}\right)\left(\frac{33}{32}\right)}\underset{d^3}{\left(\frac{13}{64}\right)^3}\underset{t}{\left(\frac{33}{32}\right)}} = .8 \text{ ksi}$$

Shear stress due to transverse forces at x-x axis

$$= \frac{VA\bar{y}}{It} = \frac{\left(\frac{.3 \times 7.5}{2}\right)\left(\frac{1}{2} \times \frac{33}{32}\right)\left(\frac{13}{64}\right)\left(\frac{1}{2} \times \frac{33}{32}\right)}{\frac{1}{12}\left(\frac{33}{32}\right)^3\left(\frac{13}{64}\right)\left(\frac{13}{64}\right)} = 16.2 \text{ ksi}$$

The allowable bending and shear stresses according to the AASHTO Specification for Highway Bridges (1) are 0.66 F_y (23.8 ksi) and 0.33 F_y (11.88 ksi). The allowable stresses are reduced in the case of open grid deck analysis to account for stress concentration due to notching, fatigue and effects of residual stress buildup.

Stress concentration due to notching is found to be 2.4 for the cross bars of typical open grids as per the formula given by Roark and Young (22). Allowable stress under fatigue for 1/2 million cycles (Case 17 of Figure 1.7.2 of Reference 1) is given as 19 ksi; however, this is reduced by 0.5 due to low quality plug welding and the absence of vertical weld between cross and main bars. Additional reduction of 20% is arbitrarily assigned in the design calculations for residual stress buildup developed during the welding process without preheating the main or cross bars. The reduction factors 0.5 and 0.2 correspond to plug welding and residual stresses will have to be substantiated through controlled laboratory work.

Invoking the abovesaid reduction factors to A36 steels, the modified allowable bending stress is 3.2 ksi for an allowable bending value of 9.5 ksi. Similarly, modified shear stress of 3.96 ksi was obtained for an allowable shear stress of 11.88 ksi even without accounting for fatigue.

Both the allowable stresses bending and shear are well below the induced stresses due to tire pressures. Hence, the existing open grid decks are concluded to be unsafe under HS20-44 loading.

The above calculations were performed for a traffic flow that was perpendicular to the cross bars. When the traffic flow is perpendicular to main or supplementary bars (or parallel to cross bars), it was found that the supplementary bars, which are smaller in size than the cross bars, were highly vulnerable to high stresses due to longitudinal forces induced by braking or acceleration of vehicles. In the absence of supplementary bars, the main bars resist bending about their weak axis due to longitudinal forces and the stresses were very high, i.e., approximately 350 ksi for 8-foot spacings between stringers.

Stress Calculations of Concrete-Filled Decks

From the data examined so far, it appears that the stresses in steel and concrete (considering the system as a standard composite section) are within the allowable limits in main bars. Top and bottom cross bars of filled grid decks play an important role in effectively providing lateral distribution of concentrated loads. Hence, the design should not only limit the average induced stresses of transverse steel within allowable values, but due attention should be given to their spacing and splicing details because these are important links for proper lateral distribution of wheel loads.

Present methods of calculating the magnitudes of horizontal shear forces between filled decks and stringers are adequate. Reliable and adequate weldings will be satisfactory in preventing weld failures provided longitudinal expansion due to corrosion or deck cracking can be eliminated through careful detailing procedures.

Load Rating Improvement Factor

The improvement factor is generally defined as the percentage change in flexural requirements between the concrete deck system and the one employing deck load reduction technique (8). This can be directly related to the increase in load ratings due to the reduction in deck loads caused by the utilization of lighter decks such as the open steel grid panels or partially filled grid panels.

Curves A, B and C of Figure 5, respectively, reveal the percentage improvement factors in flexural requirements versus span length due to the utilization of open steel grid (15 psf) panels, partially filled (40 psf) and fully filled (65 psf) grid panel in place of 8" thick regular concrete decks weighing 100 pounds per square foot. The span lengths considered, herein, are 17.42', 30', 40', 70', 100', 150', 200' and 250'. The family of curves given in Figure 5 is generated through a simple computer program for composite deck stringer systems developed at West Virginia University, Morgantown, West Virginia, U.S.A.

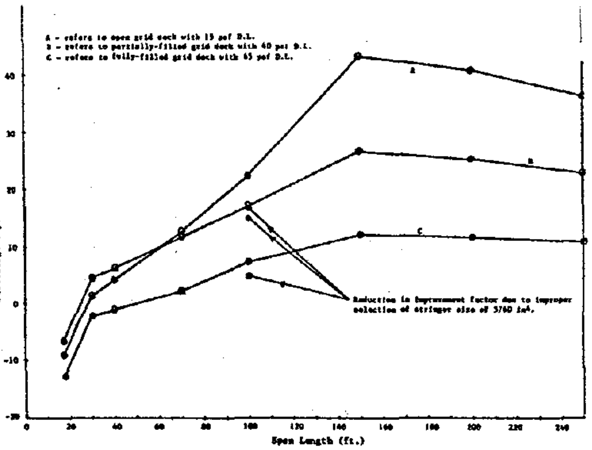

Figure 5. Span Length Improvement Factor for Dead Load Reduction Systems

In general, the percentage improvement factor, up to 100-foot spans, is below 10%, and it is even in the negative range for very short spans of 30' and 40'. This can be explained by noting that the concrete deck stiffness played a predominant role for spans below 30' or 40', in reducing bending stresses, whereas the stiffness of open- or partially-filled grid decks is not significant enough in relation to stringer stiffness to significantly reduce the bending stresses. Ratio of dead load stresses to live load stresses increases with the increase in span lengths. Hence, significant increase in the percentage of improvement factor can be noticed from the results shown in Figure 5. It is of particular interest to note that the percentage of improvement factors increased linearly with span lengths from 30' to 150', and became asymptotic for spans ranging from 150' to 250'. It is likely that one may notice a dramatic change in the percentage improvement factor between 250' and 400' spans. However, this may depend upon the type of flexural rigidity of girders and the type of superstructural system employed for transmitting truck loads. Also, it is interesting to note that an over design of stringers can dramatically reduce the percentage increase in the improvement factor. This has been demonstrated in Figure 5 by plotting three different points in open circle for a 100-foot span length. As expected, the maximum percent increase in the improvement factor corresponded to open steel grid decks and the minimum increase corresponded to fully-filled grid decks; i.e., percentage increase is indirectly proportional to the deck load. Any reduction in the improvement factor, as in curve A of Figure 5, for spans greater than 150', may be attributed to a "less than optimal" stringer flexural rigidity than has been incorporated in the calculations.

CONCLUSIONS AND RECOMMENDATIONS

This paper dealt with the first phase of the comprehensive research on grid decks wherein: (a) a carefully prepared questionnaire was sent to various highway agencies, fabricators, consultants and contractors to bring out the common maintenance and performance problems, synthesize the information to understand the causes of failures and suggest the remedies; (b) current grid deck design philosophy of AASHTO is critically reviewed; and (c) stress calculations and load rating improvement factors are provided for the reader to appreciate the present deficiencies and future potential of grid decks as bridge floor units.

More specifically, synthesis of responses to the questionnaire revealed that the open steel grid panels are economical for either rehabilitation purposes or temporary as well as permanent decking. However, a majority of the open decks are exhibiting severe maintenance problems such as the failures of cross bars, diagonals, and plug welds due to inadequate shear resistance or accidental interlocking of foreign objects, failure of welds connecting the grid deck and the stringers due to inadequate or poor quality field welds and due to overstressing under fatigue, and poor skid resistance with regard to certain grid types. Also, some difficulty in proper fitting of grid panels in the field has been reported. The majority of these problems can be easily corrected by adopting fillet welds instead of plug welds in connecting the main and secondary bars. Also, other remedial measures are suggested in Section 2.

The concrete-filled grid decks are exhibiting longitudinal growth of the deck, cracking and pitting of the concrete cover (if it is 3/4" to 1", or less), and some slipperiness and weld failures between stringers and filled decks. To eliminate or minimize the longitudinal growth of filled decks, it is suggested that the concrete fill should contain corrosion retardant admixtures (calcium nitrite, for example) with waterproofing membrane to prevent the deicing chemicals from seeping into the deck cells and corroding the steel. Also, it is suggested that adjacent grid panels be properly spliced by welding all cross or main bars, and rigidly connecting them to stringers.

Significant increase in the percentage "improvement factor" is noticed from the results shown in Figure 5 for open as well as filled grid decks. It is of interest to note that the percentage increase in this factor is linear with span lengths from 30' to 150', and the maximum percentage of such an increase is of the order of 40 to 45. From this study and others (6, 18) the grid deck-stringer systems are found to be not only highly favorable in terms of "improvement factors", but also favorable in terms of their costs.

The following recommendations are suggested for future research:

(a) Design equations derived herein are functions of trigonometric series and problem parameters. These have been simplified for design equations. Additional simplifications and validations of design equations derived by the author (8) have to be made through testing practical steel grid decks in the field as well as the laboratory;

(b) Laboratory and field testing has to be performed to further validate the causes of failures that were described earlier, and to see if the proposed modifications of alterations, such as the increase in thickness and elimination of reentrant corners in cross bars and very slow cooling process after welding the main and cross bars, are adequate for better performance of grid decks over a fifteen- to twenty-year period;

(c) Accelerated laboratory tests, to prevent fatigue failures of welds, cupping and longitudinal growth of concrete-filled decks, should be conducted to minimize long-range maintenance problems;

(d) The performance evaluation of several overlay materials under the effect of studded tires;

(e) Actual analysis of the commercially available grids has to be performed by varying the grid deck parameters; and

(f) Computer-aided software needs to be developed for routine renovation operations or for establishing the ratings of different superstructural systems with and without grid decking.

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REHABILITATION WINS OVER REPLACEMENT OF WATER TREATMENT PLANT AT FINDLAY, OHIO

Anand B. Gogate

Associate Professor Department of Architecture
The Ohio State University, Columbus, Ohio; also,
Chief Structural Engineer, Elgar Brown,
Consulting Engineers, Columbus, Ohio

SUMMARY

Most well designed structures have a virtually unlimited life if proper preventive maintenance is provided to them. In the case of water and sewage treatment plants, a common form of damage is due to abrasion of the concrete at the water line. The cost method of rehabilitation of structures and their effective use are the two most significant factors that generally govern the final decision in regard to their rehabilitation. A case history of rehabilitation of a water treatment plant at Findlay, Ohio is discussed in this paper. The overall economy of rehabilitation in comparison to reconstruction is demonstrated.

INTRODUCTION

The water treatment facility at Findlay, Ohio dates back to 1888. The original system has been improved and expanded over the years. The present (1980) facility consists of two upland reservoirs, a well field, raw water lines, low lift pumps, a lime soda softening water plant, high service pumps, water distribution system and sludge facilities.

Malcolm Pirnie Inc., Consulting Environmental Engineers of Columbus, Ohio and White Plains, New York, were commissioned to investigate the facilities and submit a critical engineering report to decide whether the facilities could be rehabilitated from an optimal standpoint. The other option to be considered was the replacement of the plant with a completely new facility. The author's firm was involved as experts in Structural Engineering in evaluation of the structural aspects and the possible deterioration and damage to the structure and the cost of possible rehabilitation.

The objectives of this paper are twofold. First, the paper reviews some of the inherent considerations that must be made in regard to rehabilitation of a process facility. Secondly, relevant data on the Findlay Plant is provided in each subsection of this paper, so that it may be viewed against the general philosophy defined in the preamble of each section and subsection.

In the case of the Findlay Plant, rehabilitation turned out to be the optimal decision and the implementation of the recommendations of the report (1) on which this paper is based are being incorporated at the present time (1981).

STRUCTURAL LIFE, EXISTING DAMAGE AND REHABILITATION POTENTIAL

Most well designed structures have a virtually unlimited life if proper preventive maintenance is provided for them. However, damage to the integrity of a structure can occur due to catastrophic or unforseen overloads such as floods, indiscriminate use, or earthquakes.(2) Overloading of a reinforced concrete member can lead to various types of structural distress. These types are:

1) Excessive flexural cracking in the tension zones.

2) Shear and diagonal tension cracking.

3) Shear-bond failure leading to side splitting.

4) Failure of beams in compression zone.

5) Failure of concrete columns due to spalling of concrete due to excessive axial load and/or end moments.

6) Large deflection.

Rational and practical procedures exist to repair all the categories of structural damage stated here with the use of epoxy agents and proper repair techniques.(3)(4)

In the case of water and sewage treatment plants, a common form of damage is due to abrasion of the concrete at the water line. This damage is particularly pronounced in regions subjected to freezing weather.(1)(3)

In the case of the Findlay Plant a very significant part of the damage to the concrete circular tanks and flumes belonged to this category. In some cases, the damage due to abrasion has caused spalling of the concrete cover and the reinforcing steel has severely corroded. However, most damage of this type can be repaired by wire brushing of the rusted surfaces of the reinforcement and removal and replacement of the steel with new reinforcing steel. The patching of the concrete can then be performed with new concrete.

A key factor in such repairs is, of course, the estimate of the future life of the structure. This magic number directly influences the decision of "to rehabilitate or not to rehabilitate".

In the case of the Findlay Plant, the types of structures to be considered are:

1) Reinforced concrete "open top" circular and rectangular settling tanks.

2) Reinforced concrete channels and flumes.

3) Reinforced concrete underground clear water storage tanks.

No structural damage due to overloading or due to catastrophic event was observed. Almost all the damage to the concrete was due to abrasion and weathering due to the high variations in atmospheric temperature. Typically, in Findlay, the low temperature in winter is in the range of 10° to 20° F below zero, and the summer high is in the range of 90° to 100° F. The common cause of damage in such an environment is due to initial cracking caused by the combined effects of shrinkage and thermal stresses. This can lead to spalling of the concrete cover due to the abrasion of the flowing water, which in turn can lead to the exposure and corrosion of the reinforcements. Some segments of the tank walls required a complete replacement by removal of the existing concrete and splicing of the existing reinforcement with new reinforcement. However, in many cases, mere patching of the surface abrasion with new concrete or epoxy is deemed adequate.

The rehabilitation potential for the structure with proper preventive maintenance is considered very good. It was concluded that the rehabilitated facility with proper maintenance could be made to survive at least sixty years in the future, i.e., 2040 A.D.

Bennington and Emmons[3] have developed a very interesting diagram (see Figure 1) which sums up the causes of tank wall damages, the effects, and certain procedures for eliminating the problem in a rehabilitated structure.

PROCESS OR FUNCTIONAL CONSIDERATIONS

The efficiency and adequacy of the process is a fundamental consideration in any rehabilitation. The rapid development of technology generally results in some compromises to be made in this regard. It is not uncommon to find a process that was considered very efficient at one time to be found not so desirable a decade later.

In the case of the Findlay Water Plant, this aspect was not a major consideration since the technology in the field of water purification has not changed very much in the last sixty years. However, certain upgrading of the system was essential. The main items in this regard with costs are enumerated in Table 1 of this paper.

ECONOMIC FEASIBILITY

Three possible alternatives were considered in the economic analysis of the Findlay Plant.[1] These were:

1) Upgrading of the plant with structural rehabilitation.

2) Rehabilitation of the structure and process to last for a 20 year period and reconstruction thereafter.

3) Construction of a new plant at a new site.

The 20 year present worth analysis of the three alternatives is shown in Table 1. Based on these figures and other analyses it was decided that alternative No. 2 was the most economically optimum for the Findlay Plant.

CONCLUDING REMARKS

The overall economy of the world has created a money crunch even in the affluent countries of yesteryears. Consequently, rehabilitation of structures has gained greater significance in the last few years on a global basis. The cost methods for rehabilitating structures and the effective use of the structure are the two most significant factors that generally govern the final decision in regard to rehabilitation of a structure.

ACKNOWLEDGEMENTS

The writer would like to thank Elgar Brown, of Elgar Brown, Consulting Engineers, and William Cummins and Glen Hughes of the Columbus, Ohio office of Malcolm Pirnie Inc., Consulting Environmental Engineers, for all their cooperation and help in developing this paper.

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TABLE 1

20-YEAR PRESENT WORTH ANALYSIS

Item	Present Worth A. Upgrading	Present Worth B. Reconstruction	Present Worth C. New Plant
Present Worth Project Costs	$16,791,000	$16,791,000	$23,084,000
Present Worth Constant Operation & Maintenance	6,879,300	6,879,300	6,189,700
Present Worth Variable Operation & Maintenance	770,200	770,200	761,800
Present Worth Salvage Value (Subtract)	(-) 1,133,500	(-) 1,057,300	(-) 1,842,900
Total Present Worth	$23,307,000	$23,383,200	$28,192,600
Annual Equivalent Cost	$ 2,737,600	$ 2,746,600	$ 3,311,500
Water Bill Increase for Avg. Customer (per month)			
Residential (1400 cf)	$ 7.93	$ 7.96	$ 10.13
Commercial (100,000 cf)	392	394	501
Industrial (1,000,000 cf)	3,717	3,733	4,746

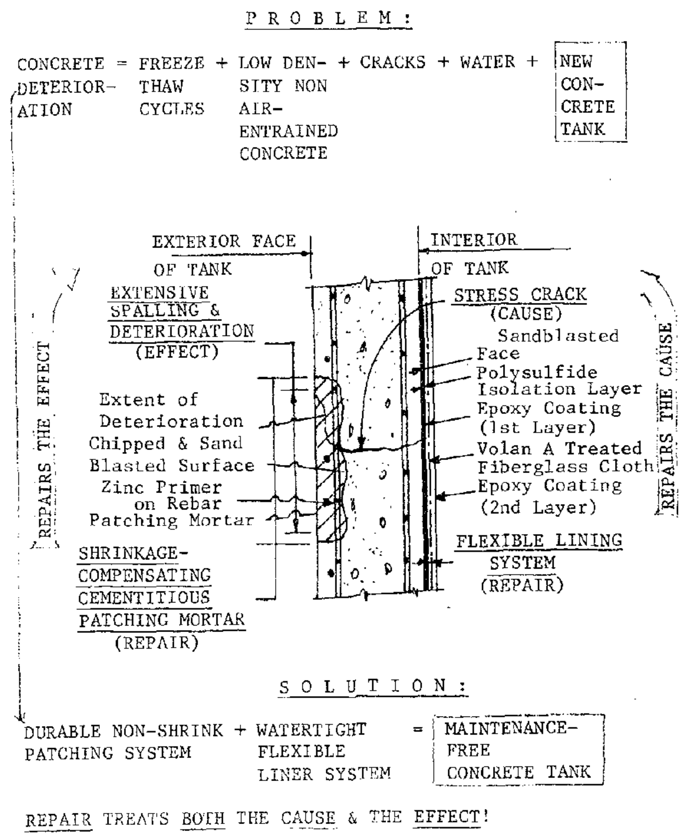


Figure 1 - Sectional Plan of Tank Wall and Repair Systems on Exterior and Interior Faces (Taken from Reference 3).

STRUCTURAL INVESTIGATION OF CLARA BARTON HOUSE
GLEN ECHO, MARYLAND

Gajanan M. Sabnis
Professor of Civil Engineering,
Howard University, Washington, D.C.

Ramesh Patel
Project Manager,
Sheladia Associates,
Riverdale, Maryland.

SUMMARY Clara Barton, a Humanitarian and the Founder of the American Red Cross, spent the last 15 years of her life in a house in Glen Echo, Maryland, near Washington, D.C. In 1889, the Red Cross Building (warehouse) was built in Pennsylvania to provide mass shelter for Johnstown flood victims. Later on it was dismantled and some of the lumber (hamlock and others) was shipped out used, in 1891, frugally to build a national headquarters for the American Red Cross which became The Clara Barton House. It is a 90 year old structure with great historic importance. The present work refers to preservation and to its rehabilitation to the present standards of safety, so that it may be enjoyed by the public as a part of a historical landmark, particularly due to the centenary of Red Cross in the United States. Extensive field investigation was carried out by a team of engineers to estimate the existing loads and to recommend methods for upgrading the structure to the BOCA - 1980 requirements.

INTRODUCTION

The Clara Barton House (Fig.1) sits on small hill or bluff located near Glen Echo Park, Maryland, a few miles north of Washington, D.C. To the south the house overlooks the George Washington Memorial Parkway and, a little further, the Potomac River. To the north it overlooks a large gravelled parking lot which is the result of a 30 feet deep excavation of what used to be the lawn in front of the house.

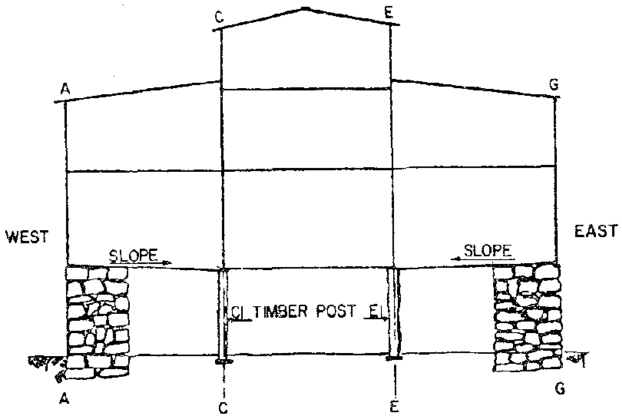
FIG. 1

FRONT ELEVATION
Fig. 2

First built in 1891, the house (Fig.2) consists of a balloon wood frame, three-story high structure resembling a Roman "Basilica" with a central "nave" and two lateral bays. The balloon frame is supported on the east and west with stone masonry walls (about 2 feet wide) at the periphery, while the central "nave" originally was supported by timber posts only. Some Concrete Masonry Unit (CMU) supports were added recently.

In balloon-frame construction (Fig.3) the wall studs and the floor joints rest on the anchored sill. Also wass studs extend from the sill of the first floor to the top plate or the end rafter of the second floor. The ends of the second-floor joists bear on a 1-by-4 inch ribbon that has been let into the studs.

In 1897 the house went through extensive remodeling which included: a change from a stone facade to a timber one with two lateral towers; the ad-



FRAMING ELEVATION
Fig. 3

dition of a simple veranda with central steps; the addition of six brick chimneys, partitions, ceiling, closets in the central bay, kitchen and other living quarters; excavation of basement earthern floor to a uniform depth of over 6 feet; the addition of basement windows and other changes like a cellar kitchen, servant bedroom and carriage room in the southern end of the central bay.

GENERAL CONDITION AND EVALUATION OF STRUCTURE

Various components of the structure and the general condition as they exist are presented in this section. The evaluation of structure is primarily based on the existing condition of structure and information from field investigation.

The foundation and basement walls are built with "Random Rubble" type stone masonry. They vary in depth from 2 to 4 feet relative to the outside grade, and often are completely exposed if not above the level of the excavated basement floor. Joints between stones vary from 1/2 to 3 or more inches and mortar is lacking in many places.

The south side of the building has no stone masonry footing except at the corners. The north side has a continuous stonewall. The east and west sides have a discontinuous foundation and stonewall, with gaps or intervals ranging from 3 to 8 feet. The central bay is supported by intermediate timber posts and CMU oukasters with their footings generally exposed.

Exterior Structure: The east side of the house seems to be more humid and therefore shows more deterioration and decay due to humidity and exposure to water for long periods than other sides. Gaps between the brick chimney and the stonemasonry tower, and openings between basement window frames and stonewalls could be observed. Wood sidings are cracked and some boards are loose. The lower part of the corner strips on the sothern corner are rotted. Gaps between stonewall sections in the tached below the window frame. This partition is in direct contact with the soil and as a result the wood is getting rotted. (Fig. 4)

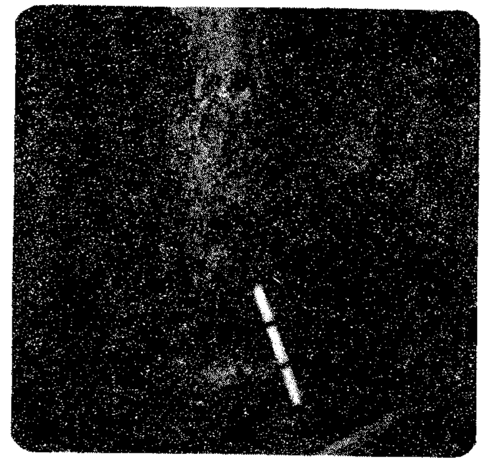


FIG. 4

Originally a stone facade, the front of the building presently exhibits a facade with a frame structure and a concrete floor veranda in front flanked by two stone towers. The frame structure facade is generally in fair condition, although around and below the main entrance there is indication of water coming down along (and inside) the wall from the terrace above (Covering the veranda). The clapboard siding, replaced recently, is slightly different from the original one still found on the east and west walls but is in good condition with no signs of termites. Some gaps are visible between the siding and the stone walls of the towers, especially at the east side corner. Some window sills are soft. Timber doric columns, originally installed around 1919 and replaced later, are still in good condition and so is the ceiling of the veranda. Stone walls below the veranda, under the columns, are cracked in places along the joint with the wall of the tower.

The east concrete steps of the veranda are in fair condition, while the west concrete steps exhibits spalling of some concrete risers and cracks on the supporting walls.

Interior Structure: The interior of the building has three levels. Several rooms are being used presently and some are blocked for historic preservation of artifacts and furniture belonging to Clara Barton. Additions, like the kitchen and toilet, are being made at the basement level to convert part of it as living quarters.

The first floor rooms are generally used as offices, meeting rooms and other facilities. The vestibule floor, constructed of timber at this level, had problems of decay and infestation of subteranean termites. It was strengthened by injecting epoxy. Some restoration work in the historic Red Cross Offices is being carried out on the south side of the building. At the second floor level, there are two apartments occupied by persons in connection with the Red Cross work.

The third floor has three rooms which have exhibits for public viewing and are blocked out from additional live loads. These rooms overlook the roofs on the east and west sides.

Roof: The roof is the Basilica type with a section shown in Fig. 3. Part of the roof is covered by a Gable type at each end, while its central portion is an elevated hip roof. The east and west side bays are covered by a shed roof. Rafters in these bays are sagging. The roof sheathing is made of 1-inch boards and covered by a sheet metal roof (replaced in 1966). The central bay is terne coated (80% lead, 20% tin), covered with stainless steel metal sheets (Fig.5).

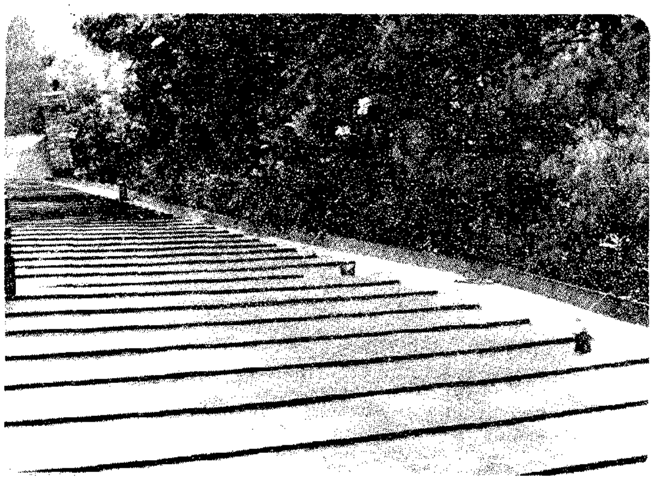


FIG. 5

DETAILS OF REHABILITATION

Along from the outside at the first outset, it appears that the structural condition of the house is good. Detailed survey and inspection revealed that three things must be done to improve it, so that the house can be preserved as an historical landmark and a tribute to Clara Barton. They are:

a. General clean-up of vegetation and landscaping.
b. Exterior structure restoration.
c. Interior structural strengthening....addition and replacement.

The general clean-up did not need a great emphasis but becomes apparent since it added to the existing problems and made the condition dangerous. Structural strengthening and replacement needed thorough structural evaluation and calculations based on the engineering knowledge and judgement on the properties of materials with which the structure had been designed and constructed. The brick sidewalk along the east and west walls, which extends from the north towers southward, is covered by ivy and other vegetation which, apart from damaging them, render them useless. The immediate surroundings seem to have more vegetation (bushes, conifers, deciduous trees bamboo) at the present time than the one during the first decades of this century. Of some concern is the vegetation growing along the west side, which is within 4 feet of the west wall, and poses a potential danger of damage to the foundations.

The original construction site was slightly graded and sloped naturally southward with a drop of 6 feet in ablut 84 feet, the original length of the house, thus giving at the front(north side) a crawl space of 3 feet and at the back (south side) a 10 feet high cellar. Such site with slope generally provides an excellent drainage for rainwater. However, the ground adjacent to the building badly needs regarding to insure proper drainage away from the stone walls. The same applies to brick aprons alongside the east and west walls. The removal of vegetation, mentioned earlier, would also keep the downspouts and vertical drains clear to prevent their clogging, thus causing the problem of overflowing of gutters alongside the walls.

Exterior Structure: Several wood frames of windows had become soft due to weathering action and siding of the windows at grade level was decayed. These had to be replaced. In general, wherever the woodwork is in direct contact with the ground proper care had to be taken to prevent any damage and further decay. In general, the stonewall needed repointing, and the decayed weather boards were replaced. The frame of the east side exterior balloon frame wall was found to be rotted and decayed in some places where checking was possible (e.g., Clara Barton Bedroom on the second floor). This suggested an in-depth inspection of the second floor exterior east wall by either removing the inside face or outside sideboards in order to check the ribbon and other members. In addition, traces of water seepage along the east and west walls on the third floor and a few wet patches on the first and second floor ceilings indicated possible decay of the balloon frame. Further investigation was necessary to confirm this.

Interior Structure: It is important that in such a structure that a thorough understanding is developed for the entire interior structure for further strengthening of it. Proper consideration had to be given to not only the actual loads on the floors but also to what loadings may be applied to some of the areas. In this section detailed structural conditions are presented, while the following two narrate the estimates on loads on the structure and also the capacities of floors and supports.

In the utility room, some planks were decayed at the grade level. The ceiling joists showed 3/4 inch to 2 inch holes. Water from the radiator pipe going through the ceiling (not far from the chimney) has leaked, and, consequently, the subfloor around it in the upper level was decayed. The nearby joist also had been affected badly by the water. A brick chimney on the east wall needed repointing since the mortar joints showed spalling and disintegrating. The joists sitting on a single header on the chimney wall have only a 2 inch bearing support (Fig. 6).

FIG. 6

Footings and supporting posts for the whole house were clearly visible in the basement since most of the basement floor has been excavated to provide a height of about 6.5 feet from the excavated floor to the lower part of the joists (Fig. 7). Supporting posts in the basement fall into two categories: timber posts and concrete masonry pilasters. Many of the timber posts were decayed and needed to be replaced. These were provided with a better foundation since in most cases they rested on very small and inadequate footings. A stud wall upto the utility room corner and a CMU column at the north partition was recommended and provided.

Fig. 7

Flooring at the basement level is generally of two types: first, the earlier type which is a floor consisting of 1/2 inch plywood over a subfloor of tongue and groove boards on 2 inch vertical X 8 inch horizontal joists at about 2'6" o/c; the joists are supported toward the ends by 2 inch X 8 inch beams. At the center, under the joists, there are 2 feet high posts (Fig.8). The other type of floor consists of original pine boards on 2 inch X 8 inch pressure treated subfloor and 4 inch concrete slab. The floor generally was in good shape; however, its support system (subfloor) for the entire floor needed strengthening and calculations had to be made to accomplish it.

FIG. 8

The timber solid posts of a portico (added in 1911) projecting 7 feet are in good condition having been replaced when the concrete slab was cast under them. One of the post, however, is slanted and had to be straightened to a vertical position. On the east side of the portico there is a 2 inch X 4 inch diagonal strut which may be removed due to its structural redundancy.

The concrete slab under the portico has two transverse cracks. One at 3'2" west of door 12, and the other 1'6" east of door 12. A gap is visible between the wood frame around windows 15 and the stone wall. Proper caulking was done to avoid leakage.

The west wall consists of stone masonry sections with some footing exposed since they are about 30 inches below the outside grade level. The footing of the CMU columns and posts were found to rest on top of the earth floor and were modified to a proper size.

Most of the first floor has exposed joists, typically 2 inch X 8 inch 2'-0" o/c which sagged and sometimes an intermediate support had been placed at mid span (line B); however, these joists were found to be inadequate and new joists were provided in between them. Some joists had lost a considerable section due to two water pipes going through them vertically. All CMU columns and timber posts supporting this floor did not have adequate footing, and since they were spaced at irregular intervals, new ones were designed and used to replace them.

The vault is paved with bricks without mortar and water has been leaking through the floor probably after heavy rains. The floor of the vault, made of two boards, had decayed at the bottom and was fixed accordingly to make its close properly. The frame and boards under windows, in general, were rotted and were replaced with concrete below external grade level and new treated boards above grade.

Upper floors were inspected for structural condition somewhat differently due to the planned nature of rooms as discussed next.

ESTIMATE OF SAFE LOADS

In absence of the original design calculations for the structure, loads on floors were estimated by an alternate approach. These estimated loads merely represent what the structure can bear safely and to what degree it needed to be strengthened. An average value for allowable stress at 1200 psi in the subfloor beams was assumed and load values (as uniformly distributed load in pounds per square feet) were obtained for the entire floor. It was clear that the entire floor needed strengthening by additional wooden joists. On the average, the maximum live load that the present floor could withstand safely is of the order of 14 pounds per square feet, compared to the present (1980) standard of BOCA, which requires 60 pounds per square feet for public buildings. Thus additional subfloor members were provided to accomplish this. The second floor balcony in the central portion was found safe for a live load of 60 pounds per square foot, and, therefore, no strengthening of that part was found necessary.

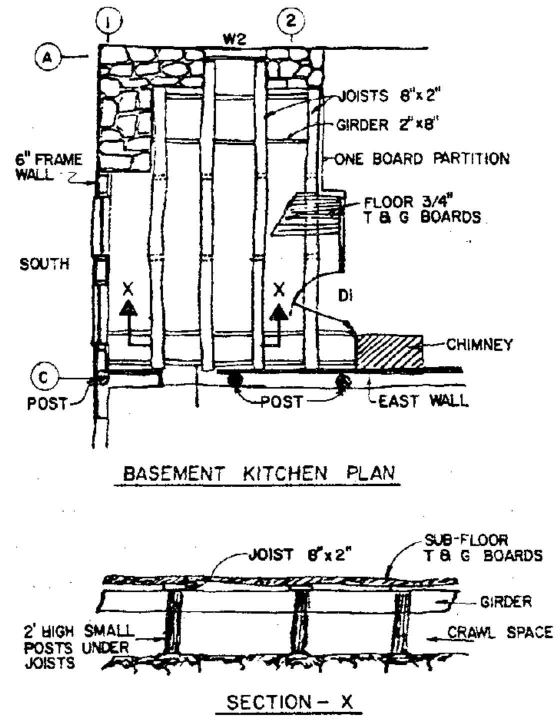


Fig. 9

ADDITIONAL COMMENTS ON STRENGTHENING

In an old but historic structure, such as the Clara Barton House, the main idea was to preserve it and to rehabilitate it to the present standards of safety, so that it may be enjoyed by the public as a part of history. A number of remedial solutions were recommended in addition to the above rehabilitation. These included:

a. Block out areas from excessive loads;
b. Repair certain areas; and
c. Specify the loads in special areas.

Since certain areas are occupied by the items of historic interests, such as furniture, files and other artifacts, they were blocked out from the general public access. This not only protects and prevents the damage that may be done to these items, but also such areas could be left without designing for heavy loads as standards specify but lighter loads for occasional use.

Some areas of floors (or subfloors) were repaired locally and fixed for use without involving a great deal of cost. This was done once a particular member had been judged to be strong by incorporating some strengthening device.

As certain space in the house was found to be marked for the use as a congregation, meeting, off-

ice or residential quarters permanently, they were considered specifying different loads in different locations. Also the maximum number of persons using certain rooms were specified so that the actual average live load may be considered rather than the standard specified by code.
When none of the above was possible, some of the members were replaced. This was kept to a minimum as it was a more difficult, expensive and time consuming process, since the integrity of the structure must be maintained during the reconstruction process with minimum inconvenience to the occupants of the structure.
The entire work was done in stages without damaging the structure or sacrificing the safety of theoccupants.
Priority for various items of work to be performed was drawn from their importance from the point of view of structural safety and also from additional deterior-ation. In the former aspect, the structure should remain functional and show more distress until repairs are performed as recommened in the next chapter, while the latter indicates that suitable precaution must be taken to prevent further deterioration. Generally speaking, the prevention of leaks, proper grading of the surroundings, trimming of trees or removal of some of them, fall in the category to prevent further damage to the structure. Such items of work had to be undertaken immediately followed by structural repairs. The structural repairs and additions of members (to strengthen the structure) were done before it was opened to the public.

UPGRADING AN EXISTING FOUNDATION OF A NUCLEAR POWER PLANT TO MITIGATE SEISMIC LIQUEFACTION

Mysore S. Nataraja
Chief Geotechnical Engineer,
Sheladia Associates, Inc., Riverdale, Maryland
(formerly, Sinior Geotechnical Engineer,
Dames & Moore, Washington, D. C.)

SUMMARY

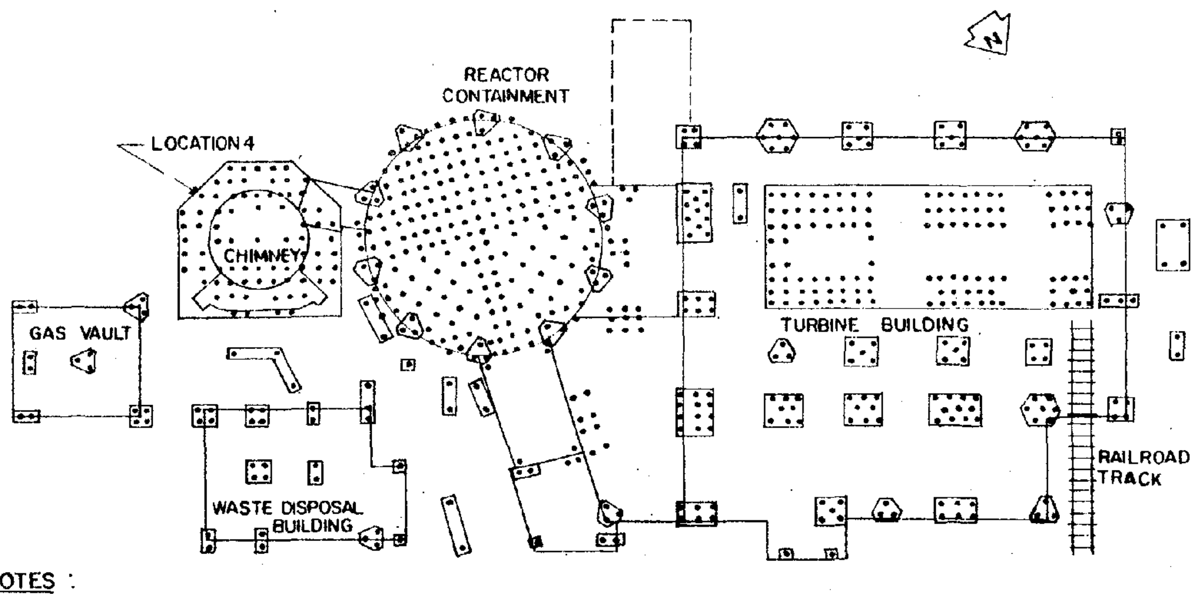
The foundation system of a small Boiling Water Nuclear Power Plant (50-MW) built in the early sixties was declared "marginal" with respect to the factor of safety against seismic liquefaction by the United States Nuclear Regulatory Commission (USNRC). Several design alternatives were considered to rehabilitate the existing foundation system to withstand the Safe Shutdown Earthquake (SSE). A dewatering system lasting for the remaining life of the plant was considered the most suitable and economical alternative under the existing conditions. This paper presents some of the salient points of the design and the potential problems associated with alternative mitigative measures available to geotechnical engineers.

INTRODUCTION

A 50-MW capacity Boiling Water Reactor (BWR) was constructed in 1962 near the village of Genoa, Wisconsin. Geologically, the site is located in the Central Stable Region of the North American Continent which is characterized by minimal seismic activity. The plant was built on about 20 feet of hydraulic fill overlying 100 to 130 feet of glacial outwash and fluvial deposits of Mississippi River Valley.

The critical structures of the plant, such as the containment building, the turbine building and the stack, rest on Raymond step tapered piles of different sizes, lengths and configurations. Figure 1 shows the piling plan for the plant. The reactor containment rests on 232 piles, the turbine building on 310 piles, the stack on 78 piles, the waste disposal building on 28 piles and the gas valult on 16 piles.

The Nuclear Power Plant and the site have functioned satisfactorily without hazzard to public health during the last 18 years of existence and the plant is expected to have a remaining useful life of some 10 years. In the meantime, the liquefaction potential at the plant site came under re-evaluation in conformance with the new regulatory requirements which are considerably more stringent than those in vogue at the time of initial construction. The main concern is the response of the soil foundation system during an earthquake. If the soil supporting the piles liquefies (that is, looses all strength and essentially behaves like a thick fluid) during an earthquake, then the plant could sink. If the soil surrounding the piles liquefies, then the piles would loose their lateral support and could buckle during seismic shaking.



NOTES :
PLANT GRADE ELEVATION IS +639 FT.
BOTTOM OF CONTAINMENT VESSEL IS ELEVATION +610 FT.

PILING PLAN
(FIGURE 1)

A series of field and laboratory investigations and analyses were performed to study the liquefaction potential at the plant site. Interpretation of the field and laboratory data yielded a range of factors of safety. Independent analyses performed by the consultants for the plant and the experts consulted by the USNRC yielded different factors of safety, even though field and laboratory data used by both the parties were the same. The differences in the analytical results were mainly due to the differences in the degrees of conservatism in the interpretation of field and laboratory data and selection of design seismic parameters such as, duration and number of cycles of shaking and ground surface acceleration resulting from the SSE. Because of the sensitive nature of the issue of public health and safety, the USNRC directed the plant to look into the mitigative measures available, to preclude liquefaction of the foundation soils and resulting collaspe of the plant structures resting on pile foundations, due to lack of lateral support. Discussions related to the alternative mitigative measures and salient details of the recommended design will follow.

Lateral Capacity of Piles

While the containment vessel foundation mat is supported on piles on a bearing layer which, even according to conservative analyses, is not liquefiable under the SSE, these piles require lateral support above this layer to prevent buckling failure. If liquefaction occurs to some depth along the piles, the lateral support will either be dimenished or totally absent. Further, as upward seepage from the liquefied zone develops, lateral support may be lost all the way from the zone to the base of the mat foundation. The Euler buckling load, P_{cr}, for a typical pile was calculated as follows:

$$P_{cr} = \frac{2.05\pi^2 EI}{L^2}$$

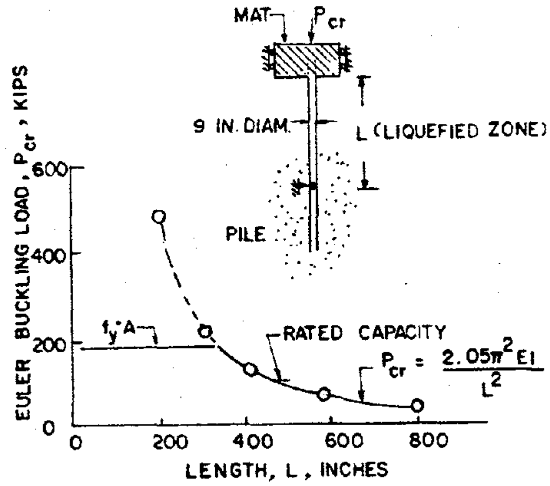
where

E = Young's Modulus of the pile

I = Moment of inertia of the cross section about its neutral axis

L = Unsupported length

An average pile diameter of nine inches was used in the calculations, even though it varied between eight to 12 inches. The piles are made of steel shell of Young's Modulus of 3,000,000 psi, filled with concrete of 3500 psi, 28-day strength. Fixity was assumed at the contact between pile and the mat foundation. For these assumptions, the relation of P_{cr} to L is shown in Figure 2.



BUCKLING RESISTANCE VERSUS UNSUPPORTED LENGTH OF PILE

(FIGURE 2)

These piles were rated to have a 50-ton static load capacity. Presumably, the piles have vertical loads considerably less than this value. At the rated load, the analysis indicates that the unsupported length at which buckling would take place is approximately 40 feet. However, the suspected zone of liquefaction has a depth of roughly 20 feet below the mat foundation. The piles appear capable of supporting their vertical working loads even if lateral support is lost in this region. However, there are also horizontal dynamic loads that act on the pile butts as a result of earthquake excitation which tend to bend the piles in themselves and make the vertical loads eccentric. This would in turn, cause further bending. Preliminary calculations suggest that the bending capacity of the piles is low with respect to the moments which might occur. (There are other unanswered questions as to the existing conditions of steel and concrete in the piles.)

Because of the lack of certainty with respect to the lateral capacities of piles during earthquakes, it was considered necessary to analyze the various alternative mitigative measures.

Mitigative Measures: The top 20 feet of hydraulic fill and the remaining 100 feet of the soils at the plant site are essentially cohesionless soils, with the exception of a layer of fine silty material at the contact between the hydraulic fill and the natural soils.

The following methods are generally used for densifying and/or improving cohesionless soils: blasting, vibratory probe, vibratory rollers, compaction piles, heavy tamping (dynamic consolidation), vibroflotation, grouting (particulate, chemical and displacement grouting), vibro-replacement stone collumns (gravel drains), freezing and dewatering.

Each of the above methods was considered for its applicability to the site where an operating nuclear power plant exists. A number of them were eliminated immediately because of some serious limitations or other restrictions imposed by existing structures at the site.

BLASTING: Successive detonation of small explosive charges can be a rapid and economical means of densification of cohesionless soils. However, the shock waves and vibrations resulting from blasting may cause local liquefaction, nonuniform displacement and remodling of soil. Because of potential adverse effects on existing structures and pile foundations at the plant site and the dangers associated with using explosives in a security sensitive area, the blasting technique was eliminated from further consideration.

VIBRATORY PROBE (TERRAPROBE): This method essentially consists of densifying a certain zone of the soil by inserting a vibrating probe to the desired depth. This method is best suited for an open, undeveloped area. Settlements will be induced because of vibrations. Potential damage to underground pipelines could result. Also, it will not be possible to densify the soils under existing structures using normal working equipment and techniques. Therefore, the Terraprobe technique was considered unsuitable for further consideration for the plant site.

VIBRATORY ROLLERS: Compaction of the soil is achieved by a self-propelled and towed vibratory roller. As this method is not effective at depths of roughly six feet below the ground surface, and the need to improve the soils at plant site exists below six feet, this mehtod was eliminated from further consideration.

COMPACTION PILES: By driving displacement piles at close spacing, densification of the soil can be accomplished. In addition to increasing the density (the increase in density may not be very significant unless the spacing is very close), the coefficient of lateral earth pressure increases and results in a considerable increase in cyclic shear strength. This is significant from the point of view of mitigation of the liquefaction problem at the plant site. Therefore, this method may be considered as an applicable option requiring further consideration. However, this method also suffers from the disadvantage that it is unsuitable where there are existing structures.

HEAVY TAMPING (DYNAMIC CONSOLIDATION): In this method, repeated impacts of a very heavy weight dropped from a height are used at predetermined spacings to achieve consolidation and densification under dynamic loadings. This method is obviously not suitable at sites with existing structures.

VIBROFLOTATION: In this method, densification is achieved by vibration of a "vibroflot" (a cylindrical penetrator about 400 mm in diameter and two meters long suspended by a crane, weighing about two tons and developing a horizontal centrifugal force at 1800 rpm). Granular backfill will be used to fill up the holes which will also be compacted by vibration. Again, this method cannot be used for densifying soils under structures.

GROUTING: This method of stabilization consists of either filling in the voids in the soil mass with cement or clay (particulate grouting), or injecting different chemicals which interact to solidify and strengthen the soil mass by the formation of a gel in their pores (chemical grouting), or stabilizing the soil mass by injecting highly viscous grout to displace the soil or by pressurized compaction (displacement grouting and compaction grouting). The displacement grouting and compaction grouting may result in pockets of saturated sand. The sand particle size at the plant site is not well suited for particulate grouting. Chemical grouting is feasible but at very high initial cost. An additional disadvantage is the extreme difficulty in reaching below the structures supported on numerous piles.

VIBRO-REPLACEMENT STONE COLUMNS: This method is similar to vibroflotation, except that the granular backfill used will be coarse gravel. The principle here is, in addition to densification, the overall effective permeability of the system will be increased so as to enable the dissipation of excess pore pressure as fast as it is generated during the earthquake. Like many other methods described earlier, this method can only be applicable to areas where there are no existing structures.

FREEZING: Freezing the ground and keeping it frozen for the lifetime of the structure could eliminate the potential for liquefaction. However, this solution is practical only for temporary rather than permanent conditions. The method also is relatively very expensive and may have the undesireable effect of freezing all the underground pipelines.

DEWATERING: The presence of water is what makes the sands susceptible to liquefaction. If the pore water is entirely removed by pumping the ground water and maintaining the water table at a lower level the potential for liquefaction is reduced under the water table also, because, when the water table is lowered, the effective stresses are increased and consequently the cyclic shear strength is increased.

After a preliminary investigation of the hydrogeologic conditions at the plant site, the dewatering option was considered a feasible option and therefore, a preliminary design of the dewatering system was prepared for presentation to the USNRC.

PRELIMINARY DESIGN OF THE DEWATERING SYSTEM

Basis for the Design

An area of approximately 240 by 160 feet in dimensions comprising of the reactor containment, the turbine building, the waste disposal building and the stack was considered for dewatering. The depth of the sandy soils suspected to be potentially liquefiable at the plant site is about 40 feet. The ground water level fluctuates between eight to 12 feet below the existing plant grade. Figure 3 shows the effect of the depths to water table below the plant grade on the factors of safety against liquefaction potential at various depths. Figure 3 also shows that lowering the water table to about 20 feet below grade would increase the factor of safety from 1.39 to 1.89 at a depth of 30 feet and from 1.55 to 1.93 at a depth of 40 feet. Therefore, the dewatering system was considered adequate if the water table could be lowered by a maximum of 12 feet from its existing level. However, the preliminary design was performed for a maximum draw down of 16 feet below the existing water table elevation that is, 25 feet below the plant grade. It is proposed to determine the final draw down depth after performing appropriate field and laboratory permeability tests.

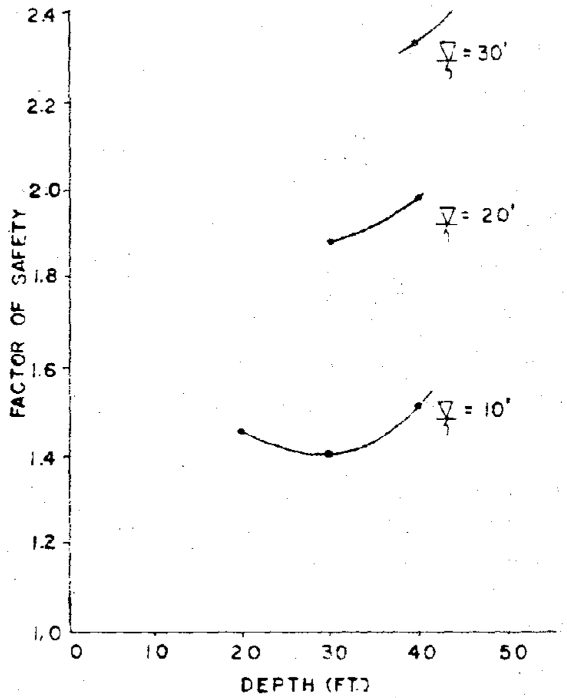


FIGURE 3

EFFECT OF DEPTHS TO WATER TABLE BELOW GRADE ON FACTORS OF SAFETY AGAINST LIQUEFACTION

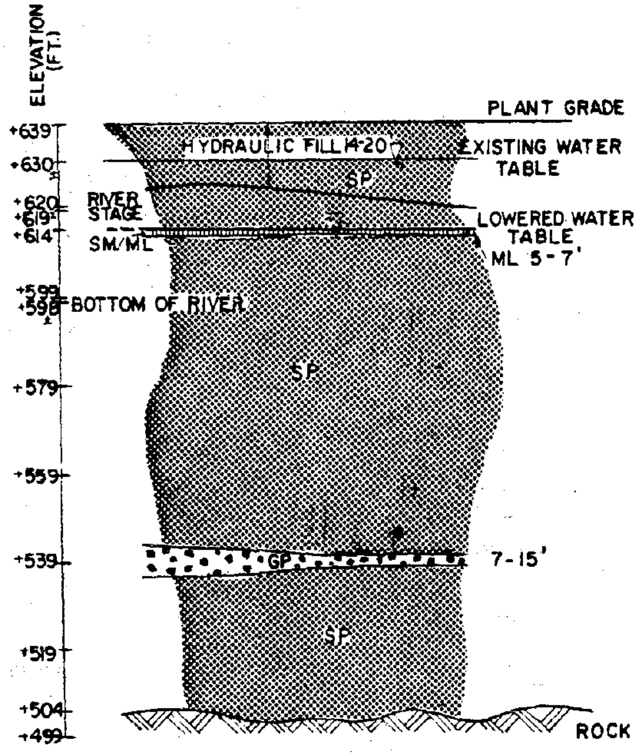


FIGURE 4

HYDRO-GEOLOGIC PROFILE OF THE SITE

Hydrogeologic Conditions at the Site

Figure 4 presents a schematic cross section of the subsoil conditions at the site. During the floods of April 1965, evidence of direct hydraulic connection between the aquifer at the site and the adjacent river was observed. The manhole covers at the plant grade (approximate elevation of 639) were forced open by the flood water and the water level of the river was also approximately 639 . The normal water level elevation of the river is 620 . The ground water level at the site fluctuates between 631 to 627. If the water table is lowered to 614 (25 feet below the plant grade), and maintained at that level, then there will be a permanent recharge from the river into the aquifer because of the direct hydraulic connection mentioned earlier.

The average permeability of the sands in the aquifer was estimated to be about 80 feet/day using the information from grain size analyses. Using some of the information provided by the site engineers regarding the quantities of water pumped out and the areas of excavation during the plant construction of the containment and the turbine building, the permeability values were back-calculated. The average resulting value of the permeability was approximately 150 feet/day. This higher value was used for the design of the dewatering system. However, pumping tests would be the basis for the final design.

The Proposed System of Dewatering

Four different systems of dewatering were considered:

1. A perimeter well point system with a central pumping station;

2. A perimeter system of suction with a central pumping station;

3. A system of individual wells with individual pumps; and

4. Ranney wells with horizontal screens.

The perimeter well point system was considered unsuitable because of the close spacings (approximately 1-1/2 feet apart) mandated by the design calculations. The perimeter system of suction wells was also considered unsuitable because of practical limitations on pumping head. Ranney wells with horizontal screens require special construction techniques and therefore were not considered further. The proposed system is one of individual wells with individual pumps connected to a common discharge pipeline, as shown in Figure 5.

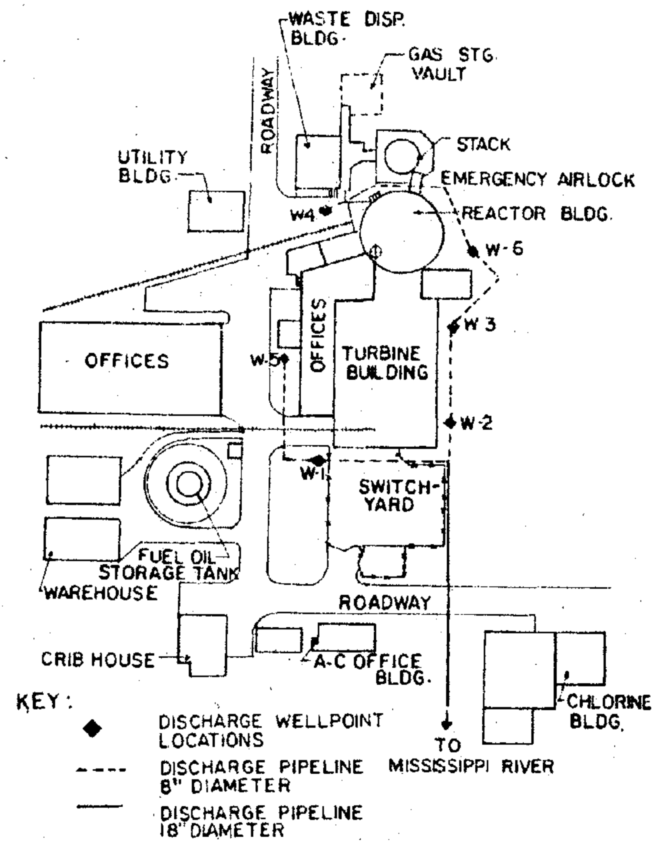


FIGURE 5. PLOT PLAN SHOWING WELL LOCATIONS

Preliminary Design Details for the Proposed System

It has been estimated that a total flow of 5000 gpm will have to be pumped continuously in order to achieve a draw down of approximately 15 feet at the center of the dewatering system. It is estimated that five individual wells, W-1 through W-5, each mounted with a 1000 gpm pump, as shown in Figure 5, will perform the required dewatering function. A sixth well, W-6, is provided as a backup for the dewatering system. The following are the design details of the wells and the pumps.

Wells

Depth	100 feet
Minimum diameter of the hole	20 inches
Minimum diameter of casing	12 inches
Minimum diameter of screen	12 inches
Minimum length of screen	60 feet
Maximum length of blank casing	40 feet
Type of screen	Johnson Stainless Steel (or equivalent).
Gravel pack	Proper size and quantities to fill in the annular space of the wells.

Pumps

Moretrench American Corporation's pump No. 10CTS or equivalent is proposed. This pump fits into a 12 inch casing and will pump 1000 gpm at 70 feet of head, and has a 20 horsepower submersible motor.

Discharge Pipelines

It is proposed that the dewatering system connected by two sections of eight inch pipelines totalling 450 feet in length discharge into an 18 inch pipeline about 120 feet in length which picks up the combined flow and discharges into the river, as shown in Figure 5.

DISCUSSION

Preliminary evaluation suggests that the proposed dewatering system could mitigate liquefaction potential at the plant site during Safe Shutdown Earthquake conditions.

There are other ramifications of dewatering, two were considered in this study. The first is the settlement resulting from increased effective stresses; the second is the down drag on the piles resulting from the settlement. Preliminary calculations show that the settlement resulting from dewatering of a limited area is about a quarter of an inch. The dewatered area is expected to experience this settlement uniformly and most likely will not adversely affect the foundations and structures at the site.

The down drag effects on the piles as a result of the settlement is expected to increase the load carried by the piles. Preliminary calculations indicate that the increase of vertical load on piles is small. This increase, it is felt, is not large enough to cause any concern on the safety of the foundations and the structures at the site.

During field investigations to study the condition of foundations it was observed that the hydraulic fill had settled considerably over the years and created a void between the bottom of the mat foundation and the surface of the hydraulic fill. Voids ranged between four inches to 12 inches in thickness in different areas of the Turbine building. The areal extent of the voids was found to be limited to a certain area of the building by performing a meticulous drilling program. These voids were subsequently filled by a grouting program.

The grouting program in combination with the proposed dewatering program is believed to constitute a reliable rehabilitation of the existing pile foundation system to withstand the seismic forces during an SSE.

SUMMARY AND CONCLUSIONS

Several alternative mitigative measures available for precluding seismic liquefaction are discussed. Limitations of such measures under the constraints of existing plant structures and foundations are described. It is concluded that a permanent dewatering system is not only feasible, but is an economical alternative under certain conditions. However, such measures are applicable only when the initial costs of installation and long term operating costs of pumps are not excessive.

ACKNOWLEDGEMENTS

The author wishes to thank Mr. Richard E. Shimshak, Plant Superintendent (Dairyland Power Cooperative, La Crosse Boiling Water Reactor, Genoa, Wisconsin) for his permission to present the findings of the study. The work discussed here was performed while the author was an acting Principal-In-Charge at Dames & Moore, Washington, D.C.

INSPECTION AND TESTING REQUIREMENTS OF CONCRETE STRUCTURES FOR NUCLEAR POWER PLANTS

D. S. Mehta

Engineering Specialist, Bechtel Power Corporation, Gaithersburg, Maryland.

SUMMARY

This paper provides a brief summary of the inspection and testing requirements for various construction materials used for the construction of safety related concrete structures. This paper also describes the inspection and testing requirements established by the various codes representing many licensing and inspection authorities, consultants, contractors, manufacturers or suppliers of the various materials such as concrete ingredients, reinforcing bars, mechanical splices, stainless steel liner plates, prestressing systems and welding materials. This paper also provides USNRC Regulatory Guides, and Code of Federal Regulations.

INTRODUCTION

The inspection and testing requirements for safety related concrete important to public safety are described in USNRC Regulatory Guides, Code of Federal Regulations and National Standards. National Standards are developed by members of working group from the American Concrete Institute (ACI), the American Society of Mechanical Engineers (ASME), the American National Standard Institute (ANSI) and the American Society of Testing Materials (ASTM).

Appendix A to 10CFR Part 50 (Code of Federal Regulations), "General Design Criteria for Nuclear Power Plant" requires that structures important to safety be designed, fabricated, erected and tested to quality standards important for the safety functions to be performed. The regulatory guides describe an acceptable method if implementing these criteria with regard to the testing and inspection of Category I concrete structures. These regulatory guides are issued by USNRC. Appendix A provides a list of USNRC Regulatory Guides and Code of Federal Regulations applicable for testing and inspection of construction material.

The testing inspection requirements specified in these regulatory guides are generally based on current industry practices. Appendix B lists national standards, and/or codes, related to inspection and tests of material. Appendix C provides cross reference to various national standards and regulatory guides.

USNRC REGULATORY GUIDES

Regulatory Guide 1.10 provides procedure for visual inspection, tensile testing, tensile test frequency and substandard tensile test results for reinforcing bar mechanical splices.

Regulatory Guide 1.15 provides regulatory position for yield strength, tensile strength, acceptance criteria and deformation inspection for reinforcing bar for safety related concrete structures.

Regulatory Guide 1.18 describes an acceptable method with regard to initial structural acceptance test, which demonstrates the capability of a concrete primary reactor containment to withstand postulated pressure loads.

Regulatory Guide 1.35 provides inspection program for prestressed concrete containment structure. It includes sample selection (number of tendons and inspection period), visual inspection, prestress monitoring tests, tendon material tests and inspection, inspection of filler grease and acceptable criteria for ungrouted tendons in prestressed concrete containment structure.

Regulatory Guide 1.55 provides designer's and constructor's requirement for placement of reinforcing bars, location of embedded item, location of construction joints and placing of concrete for safety related structure. This guide also includes a list of standards, codes, paper and other references which are generally directed toward the quality placement of concrete.

Regulatory Guide 1.69 endorses the industry standard ANSI N 10.16 as an acceptable standard for the construction of radiation shielding structures with several additional clarification included in the guide.

Regulatory Guide 1.90 describes bases acceptable to the NRC for developing an appropriate surveillance program for prestressed concrete containment structure with grouted tendons. Inservice inspection program includes force monitoring of ungrouted test tendons, visual examination of structurally critical area or anchorage assemblies and periodic reading of instruments for prestress level determination and deformation determination.

Regulatory Guide 1.103 covers the generic qualifications for BBRV, VSL, stress steel, freyssinet post-tensioned prestressing system (wire, strand, bar) used in reactor containments and reactor vessels.

Regulatory Guide 1.107 provides minimum quality standards when portland cement grout is to be used for the corrosion protection of prestressed steel for portland cement, fine aggregate, water, admixtures, physical properties of grout, duct and equipment for grouting.

Regulatory 1.136 (Revision 1, October 1978) provides requirement and guidance for material used in the concrete containment of nuclear power plant. This guide refers to Article CC-2000 of the ASME BP & V Code, Section III, Division 26 Code with few exceptions. Article CC-2000 provides testing and inspection requirements for concrete, concrete materials, reinforcement system, prestressing system, liner and welding material.

Recently revised Regulatory Guide 1.136 (Revision 2, June 1981) withdraws Regulatory Guides 1.10, 1.15, 1.18, 1.19, 1.55 and 1.103 for new applications docketed after May 1981. The regulatory positions of these guides are now considered to be covered by one or more of the following national standards:

- ACI 359 (ASME Section III, Division 2 Code)
- ACI 349
- ANSI 45.2.5

However, withdrawal of these guides does not alter any existing licensing commitments docketed prior to May 1981.

Regulatory Guide 1.142 refers to ACI standard 349 for testing and inspection requirements for safety related concrete structures other than containments.

CODE OF FEDERAL REGULATIONS

The rules and regulations of the NRC are contained in the following Code of Federal Regulations related to inspection and testing requirements for safety related concrete structures.

10CFR Part 50, Appendix A "General Design Criteria for Nuclear Power Plants"

10CFR Part 50, Appendix B "Quality Assurance Criteria for Nuclear Power Plants and Reprocessing Plants"

10CFR Part 50.55(a) "Codes and Standards"

10CFR Part 50.55(e) "Reporting Criteria for Significant Deficiencies"

10CFR Part 24 "Reporting of Defects and Noncompliance"

The Office of Inspection and Enforcement assures that nuclear facilities comply with NRC requirements specified in Code of Federal Regulations, Regulatory Guides, and National Standards included in Safety Analysis Reports. The inspection program also directs NRC inspection of architect-engineer firms, vendor and independent supplier of major materials for the construction of nuclear power plants.

NATIONAL STANDARDS

ANSI N 45.2.5 "Supplementary Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Steel, Soils and Foundations during the Construction Phase of Nuclear Power Plants". This standard provides requirements for inspection and testing of concrete aggregates, cement, admixtures, reinforcement and grout. Table A provides required test and test method for "Qualification Tests" which are performed to qualify the basic material source or manufacturer. These tests are mandatory unless current documentary test data is available to establish complete confidence in conformance to specification requirements. Table B provides requirement for test method and test frequency for "In-process Tests". These tests are performed during the course of construction to determine compliance with specified requirements and to maintain control of structural materials. These tests must be taken from the lot or batch of materials supplied and used at the site of construction. This standard is endorsed by Regulatory Guide 1.94.

ACI 359 (ASME Section III, Division 2), "Code for Concrete Reactor Vessels and Containments". This code provides requirements for fabrication, construction, testing, examination, inspection and structural integrity test of concrete containment structures. It includes requirements for concrete, concrete material, reinforcing system, prestressing system, liner plate and weld metals. This code is also endorsed by Regulatory Guide 1.136. This code established requirements for testing and establishes requirements for testing and inspection closely paralling the criteria of 10CFR Part 50, Appendix B and ANSI N 45.2.5 Standards. Until sufficient experience has been accumulated with the use of this code, instead of referencing the code in the Code of Federal Regulations, NRC Staff has endorsed the acceptability of the code for licensing purpose in the Regulatory Guide 1.136.

ACI 349, "Code Requirements for Nuclear Safety-Related Concrete Structures" Testing and inspection requirements for aggregates, cement, metal reinforcement and admixtures are specified in this code by referencing various ACI and ASME Standards. This code is also endorsed by Regulatory Guide 1.142.

Other National Standards by ACI and ASTM are referenced in the Table I and Table II.

CONCLUSIONS

The inspection and testing requirements for materials used for safety-related concrete structures have been established by USNRC Regulatory Guides, Code of Federal Regulations and National Standards. Endorsement to National Standards by Nuclear Regulatory Commission to establish standardized criteria will be extremely beneficial for the Nuclear Standardization program.

APPENDIX A

Code of Federal Regulations

10CFR Part 50 Appendix A "General Design Criteria for Nuclear Power Plants"

10CFR Part 50 Appendix B "Quality Assurance Criteria for Nuclear Power Plant and Fuel Processing Plants"

10CFR Part 50.55(e) "Reporting Criteria for Significant Deficiencies

10CFR Part 21 "Reporting of Defects and Noncompliance

10CFR Part 50.55(a) "Codes and Standards

NRC Regulatory Guides

1.10	Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures
1.15	Testing of Reinforcing Bars for Category I Concrete Structures
1.18	Structural Acceptance Test for Concrete Primary Reactor Containment
1.35	Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containment Structures
1.55	Concrete Placement in Category I Structures
1.69	Concrete Radiation Shields for Nuclear Power Plants
1.90	Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons
1.94	Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants
1.103	Post-Tensioned Prestressing Systems for Concrete Reactor Vessels and Containments
1.107	Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures
1.136	Material for Concrete Containments
1.142	Safety Related Concrete Structures for Nuclear Power Plants (other than Reactor Vessels and Containments)

APPENDIX B

National Standards, Codes, Specifications or Recommended Practice

Standard Number	Title
ACI-ASME 359 (ASME Section III, Division 2)	Code for Concrete Reactor Vessels and Containments
ACI 349	Code Requirements for Nuclear Safety Related Structures
ANSI N 45.2.5	Supplementary Quality Assurance Requirements for Installation and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants
ANSI N 101.6	Concrete Radiation Shields
ACI 318	Building Code Requirements for REinforced Concrete
ACI 301	Specification for Structural Concrete Buildings
ACI 305	Recommended Practice for Hot Weather Concreting
ACI 306	Recommended Practice for Cold Weather Concreting
ASTM A 615	Standard Specification for Deformed Billet Steel Bars for Concrete Reinforcement
ASTM A-421	Uncoated Stress Relieved Wire for Prestressed Concrete
ASTM A-416	Uncoated Seven Wire Stressed Relieved Strand for Prestressed Concrete
ASTM A-722	Uncoated High-Strength Steel Bar for Prestressing Concrete
ASTM A-370	Standard Methods and Definition for Mechanical Testing of Steel Products

APPENDIX C

Cross Reference to National Standards/Regulatory Guides

USNRC Regulatory Guide Number	Reference to National Standards, Codes or other Regulations
1.10	ASTM A-370 ACI-318
1.15	ASTM A-615 ASTM A-370
1.18	None
1.35	Applicable ASTM Standards
1.35	ACI 301 ACI 305 ACI 318 ACI 306
1.54	ANSI N101.4
1.69	ANSI N10.16
1.90	ASME Section III, Div. 2 Code
1.94	ANSI N 45.2.5 ASME Section III, Div. 2 Code
1.103	ASME Section III, Div. 2 Code
1.107	ASME Section III, Div. 2 Code Article CC-22432
1.136	Article CC-2000 g of ASME Section III, Div. 2 Code
1.142	ANSI N 45.2.5 ACI-349 USNRC Reg. Guides 1.54 and 1.94 ANSI N 101.4, ANSI 101.6

TABLE I
QUALIFICATION TESTS

MATERIAL	TEST & TEST METHOD
Concrete Aggregates (Regular)	Compliance with ASTM C-33
Concrete Aggregates (Heavy)	Compliance with ASTM C-637
Concrete Aggregates (Light)	Compliance with ASTM C-330
	As referenced in ASTM C-33, C-637 and C-330 respectively
Cement	Compliance with ASTM C-150
	As referenced in ASTM C-150
Admixtures	Compliance with ASTM C-260 or C-494, whichever is applicable
	Manufacturer's Certification
Fly Ash and Pozzolans	Compliance with ASTM C-618
	As referenced in ASTM 618
Water & Ice	Compliance with specs. for effect on: Compressive Strength Setting Time Soundness
	ASTM C-109 ASTM C-191 ASTM C-151
Liquid Membrane Forming Curing Compounds	Compliance with ASTM C-309
	As referenced in ASTM C-309
Sheet Materials for Concrete Curing	Compliance with ASTM C-171
	As referenced in ASTM C-171
Concrete Mixes	Compliance with ACI 211
	As referenced in ACI 211
Preplaced Aggregate Concrete	Compressive Strength
	CRD-C-84
Reinforcement	Physical properties of full section test specimen per ASTM A-615
	One full section test in accordance with ASTM A-370 for each bar size
Grout for Preplaced Aggregate Concrete	Flow Expansive Characteristics Bleeding Characteristics Water Retention and Unit Weight
	CRD-C-79 CRD-C-81 CRD-C-82 CRD-C-80

TABLE II
REQUIRED IN-PROCESS TESTS

Material and Requirement	Test Method & Frequency
CONCRETE	
Mixer uniformity	ASTM C-94 Initially and every 6 months thereafter
Sampling method	ASTM C-172
Compression cylinders	ASTM C-31
Compression cylinders-pre-placed aggregate concrete	CRD-C-84
Compression strength	ASTM C-39 2 cylinders for 28-day test from each 100 cu. yd. or a minimum of 1 set/day for each class of concrete
Slump	ASTM C-143 First batch produced each day and every 50 cu. yd. placed
Air content	ASTM C-173 or C-231 With each set of compression cylinders
Temperature	First batch produced each day and every 50 cu. yd. placed
Unit weight/ yield	ASTM C-138 Daily during production*
Unit weight for structural light-weight concrete	ASTM C-667 Daily during production
GROUT	
Compressive Strength	ASTM C-109 (for expansive grout use CRD-C-589) Daily during production
GROUT FOR PREPLACED AGGREGATE CONCRETE	
Time of set	CRD-C-82 Daily during production
Flow	CRD-C-82 Daily during production
Expansion and Bleeding	ASTM C-109 (for expansion grout use CRD-C-589) Daily during production

(continued)

TABLE II
REQUIRED IN-PROCESS TESTS
(continued)

AGGREGATE		
Compliance with requirements for:		
Gradation	ASTM C-136 Daily during production*	
Moisture content	ASTM C-566 Twice daily during production	
Material finer than No. 200 sieve	ASTM C-117 Daily during production	
Unit weight of aggregate	ASTM C-29 Daily during production*	
Fixed water and iron content of aggregates only for radiation - shielding concrete	ASTM C-637 Daily during production*	
Organic impurities	ASTM C-40 Daily during production*	
Flat and elongated particles	CRD-C-119 Monthly during production*	
Lightweight particles	ASTM C-123 Monthly during production*	
Soft fragments	ASTM C-235 Monthly during production*	
Specific gravity & absorption	ASTM C-127 or ASTM C-128 Monthly during production*	
Los Angeles abrasion	ASTM C-131 or ASTM C-535 Every 6 months*	
Potential reactivity	ASTM C-289 Every 6 months*	
Soundness	ASTM C-289 Every 6 months*	
WATER & ICE		
Compliance with project specifications for effect on:		
Compressive strength	ASTM C-109	Monthly
Setting time	ASTM C-191	Monthly
Chlorides	ASTM D-512	Monthly
Total solids	ASTM D-1888	Monthly
ADMIXTURES		
Chemical composition, Ph, and specific gravity	ASTM C-494 Composite of each shipment	

TABLE II
REQUIRED IN-PROCESS TESTS
(continued)

FLY ASH & POZZOLANS	
Chemical & physical properties per ASTM C-618	ASTM C-311 As specified in ASTM C-311
CEMENT	
Standard physical & chemical properties	ASTM C-150 As specified in ASTM C-183
REINFORCING STEEL	
Physical properties of full section test specimen per ASTM A-615	ASTM A-370 One full section test for each bar size for each 50 tons or fraction therof from each test

*These test frequencies shall be considered minimum unless current documentary test data are available to establish complete confidence in conformance to specification requirements

PART III

REHABILITATION OF BRIDGES AND RELATED APPLICATIONS

STRUCTURAL ENGINEERING EARTHQUAKE DAMAGE EVALUATION OF EXISTING BUILDINGS

Gary C. Hart
Englekirk and Hart Consulting Engineers, Incorporated, and Professor of Engineering, University of California, Los Angeles, California.

Robert Del Tosto
Englekirk, Hart and del Tosto Consulting Engineers, Incorporated, Rome, Italy.

Robert E. Englekirk
Englekirk and Hart Consulting Engineers, Incorporated and Adjunct Associate Professor of Engineering, University of California, Los Angeles, California.

ABSTRACT

The evaluation of the expected damage to buildings and other structures which can result from the future occurrance of natural hazards such as earthquakes is one of the most important responsibilities of the structural engineer. Structural engineers, to date, have commonly ignored this responsibility. When the structure has been evaluated, the approach taken has not usually been impressive from either a technical or professional standpoint.

This paper addresses the damage evaluation problem. We seek to present a new view of the subject which we hope will provide a productive framework for professionals interested in this area and a more accurate estimation of future damage.

INTRODUCTION

The evaluation of the expected damage to buildings and other structures which can result from the future occurrance of natural hazards such as earthquakes is one of the most important responsibilities of the structural engineer. Structural engineers, to date, have commonly ignored this responsibility. When the structure has been evaluated, the approach taken has not usually been impressive from either a technical or professional standpoint. There are many reasons why damage evaluation has not been properly addressed. We believe the single most important reason has been the lack of a clear vantage point from which the structural engineer could view the overall problem. We hope that this paper will provide such a vantage point and enable the reader to see the value of highly sophisticated research and the necessity of assimilating such research into damage evaluation.

Damage evaluation is particularly challenging because in order to obtain accurate answers, the involvement of two very different groups of professionals is necessary. On the one hand, are the researchers who are seeking a basic understanding of the natural hazard and the most accurate methodology for estimating its effects on structures. In earthquake engineering, valuable research is being done in many diverse areas: fault plane fracture mechanics; soil-building interaction; nonlinear dynamic analysis using the computer; system identification; and fuzzy set theory. The research in these and other areas is making significant contributions to our understanding of how the real physical world responds to an earthquake occurrance. On the other hand, are the practicing structural engineers who appreciate the difference between the building plans, specifications; associated analytical design models, and real world structures. We can not place a higher value on either group because each of their contributions is critically important to accurate damage evaluation.

We view the damage evaluation process as being comprised of the following parts:

1. Identification of possible modes of structural failure.
2. Selection of an analytical model of the structure that can represent the failure mode.
3. Quantification of the mean values and uncertainty corresponding to failure mode loading and resistance (i.e. demand and capacity).
4. Calculation of the value of the Reliability Index corresponding to each identified failure mode.
5. Use failure mode Reliability Index values and decision theory techniques (e.g. damage matrices) to evaluate the potential damage to the structure.

We hope that the remaining text will clearly present some of our ideas regarding damage evaluation.

PROFESSIONAL PARTICIPANTS IN A STRUCTURAL ENGINEERING DAMAGE EVALUATION

Table 1 provides a general overview of the participants who have and will make valuable contributions to a rational estimate of building damage. Within any single class of participants, there are contributions which range from highly scientific to highly philosophical. This section provides a summary of the contributions, current and future, which these participants have, or, will make relating to building earthquake damage evaluation.

The geotechnical consultant provides a description of earthquake ground motion. Research in this area, especially during the last two decades, has provided a much clearer vision of the mechanics of earthquake motion. Probabilistic and statistical concepts have gained wide acceptance and are a standard part of ground motion characterization. Much of this recent insight is the

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* Englekirk and Hart Consulting Engineers, Incorporated, and Professor of Engineering, University of California, Los Angeles, California.

** Englekirk, Hart and del Tosto Consulting Engineers, Incorporated, Rome, Italy.

*** Englekirk and Hart Consulting Engineers, Incorporated, and Adjunct Associate Professor of Engineering, University of California, Los Angeles, California.

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result of the acquisition of numerous earthquake strong motion records. The contributions made by instrument manufacturers, location selection teams, maintenance teams, and data analysis groups has been noteworthy. Future research will most certainly advance the geotechnical consultants understanding and improve their accuracy of ground motion description. It is also clear that the future will show an ever increasing use of probabilistic and statistical concepts. In addition, the estimation of peak ground acceleration will diminish in importance as a final work product. It will be replaced by response spectrum and other more meaningful descriptive items.

Code developers and system reliability ananlysts have provided contributions that are essential to understanding and estimating building response. A common misconception of code developers is that they are individuals who sit in smoke filled rooms and blithely vote on the values of coefficients associated with equations which finally appear in the code. The work by these participants can be of an exceptionally high quality. The analytical tools and theoretical concepts which have been used, especially in the last decade, are powerful. It is perhaps not appropriate to identify these two particular subgoups within the group of participants. However, it is necessary because code developers incorporate the techniques and work products include first order second moment reliability theory, a monte carlo simulation uncertainty in material properties and many other items. Many key individuals are, in fact, highly qualified in both code development and system reliability analysis. It is clear from current trends in this area that future work will result in more accurate characterization of uncertainties in both material properties and design equation accuracty as it relates to real world buildings. This characterization is, and will continue to be, done using the theory of probabilityand statistics. However, a major, and very fundamental, direction has recently been adopted. This involves the use of a <u>reliability index</u> to quantify the adequacy of proposed <u>code formulas and formula</u> parameter values. This index provides two very fundamental benefits. It enables the strength of building components to be viewed on a material by material basis and it also provides for and encoueranges the characterization of strength to be done independent of load factors. In addition, it also enables one to address safety and reliability without direct quantifying the probability of system failure. It might be noted that the use of the term probability of failure has inherent in it certain objectionable connotations. These have placed unnecessary barriers in the path of progress of rational code development.

The reliability index also enables one to draw upon past design experience. We can eavluate past design practice and quantify values of the reliability index which are acceptable based upon base performance. The reliability index is a fundamental component in our damage evaluation methodology.

Experimentalists and system identification analysts provide a bridge between laboratory and large scale test data, analytical structural mechanics and design equations. The quantification of the uncertainty associated with the ability of analytical formulas to estimate real structural response is the basic goal of these participants. With laboratory experiments on building component members, it is possible to accurately establish the material properties, dimensions, geometric parameters and component support conditions. Therefore, laboratory tests are especially important in correlating mean values of the ratio, or equation predicted response/test data response. System identification, or more accurately called parameter estimation methods, are essential in this area. Two research developments in the last decade are important as they relate to building earthquake damage estimation. First, research in the area of quantifying the material strength of existing buildings has made significant advances in the last decade. The ability of existing testing laboratories to take numerous core samples and coupon specimens for testing has been reduced in cost with a considerate increase in accuracy. The interpretation of these sample tests results have also advanced as experience has been gained. Second, research in the ambient vibration approach to quantifying natural frequencies, mode shapes, and even damping, has been significant. As a result of this research, it is possible to obtain the professional services of commercial companies to perform ambient vibration tests and to obtain, for a specific building, estimates of the building's natural frequencies, mode shapes and damping. These test results enable the structural engineer to improve the accuracy of his analytical building model. The important value and interdependance of system identification, experimental data acquisition and analysis, and, its general role in damage estimation is clear. The future of experimental testing and its application into the area of describing building characteristics is extremely bright. The last two decades have demonstrated the advances made in solid state electronics and the relatively stable costs of instrumentation. Any damage evaluation methodology must recognize this and be able to benefit from these advances.

Micro-computers, in the next decade, may soon become as numerous in structural engineering design offices as typewriters. The impact of this can not be underestimated. It relates not only to structural design analysis, but, to damage estimation. The cost of a first class micro--computer system is approximately the cost of a new car, that is, a Toyota, not a Rolls Royce. Therefore, these systems can, and, are being purchased by structural engineering design offices. In the last two decades, the impact of the computer on building structural design fell short of most predictions. The reasons for this are a matter of speculation. However, one important reason was the inability of most structural design offices to purchase their own computer. However, it is evident that the microcomputer is especially well suited to structural engineering. Evidence of this is the significant number of design and analysis computer codes which are currently operational on micro-computers. The future will show an immense expansion in the structural modeling capabilities using micro-computer programs. These structural models will enable the calculation of component member forces to a far greater degree of accuracy than simple non-computer design models. One additional benefit of micro-computers, as it relates to our damage evaluation methodology, is its compatibility with Monte Carlo Simulation studies. In it's most basic sense, a Monte Carlo Simulation study provides an uncertainty or statistical characterization of structural response. It does this by repeatedly solving the standard analysis problem and by then calculating simple statistical properties of the results (e.g. mean, standard deviation, histogram). In addition, a Monte Carlo Simulation study does not require the structural engineer to perform those integrations which are required in most other uncertainty or reliability analysis techniques.

To summarize, it is clear that a structural engineering damage evaluation methodology must be able to incorporate advances made by the noted participants. It must be formulated in a way which provides maximum flexibility for the creativity of these professionals.

ILLUSTRATIVE EXAMPLE

We now illustrate the basics of our methodology by using an example of the damage evaluation of a one story unreinforced masonry building. It is intended to demonstrate some of the basic ideas of our methodology and to illustrate the difference between our approach and a methodology currently used by HUD. The senior author of this paper, who was also an author of the HUD report, views this new methodology as the logical successor to the HUD methodology.

We hope that after the reader has studied this paper, the advantages of our damage evaluation methodology over other methodologies will be apparent. We reiterate that the final step in the evaluation process requires a subjective evaluation by the professional structural engineer. This may trouble some because of distrust of professional input or potential lack of simple numerical checks. We believe that this subjective evaluation can, if desired, be more structured in the future as more reserach and application experience is gained. Also, we believe that our methodology requires a very detailed quantification of assumptions because for each random variable, it's PDF must be defined. In addition, the final result is a single numerical value or set of numerical values for the reliability indices of the failure modes.

The examples we have selected is representative of one mode of failure considered for unreinforced masonry structures. The damage evaluation of such buildings is of considerable interest. We hope that, by working this example, the reader can also gain some insight into the building rehabilitation problem. We believe that this is important because rehabilitation/damage evaluation problems are highly interrelated.

Figure 1 shows a plan and elevation of a one story unreinforced masonry building. All walls are 9 inch double width brick and the roof diaphragm is assumed to be wood. This building is assumed to be located in Long Beach, California, and the ultimate UBC base shear force is

$$V_{UBC} = 2\ (ZICSKW) = 0.372\ W \qquad (1)$$

where

W = dead weight

Table 2 gives the shear wall shear stresses that were calculated using this ultimate UBC base shear force.

The base shear force based on an earthquake response spectra analysis for this building can be written

$$V = S_a W \qquad (2)$$

where S_a is the earthquake spectral acceleration.

Therefore, using Equation (1) and (2), we can write

$$V = S_a W = S_a (V_{UBC}/0.372) = 2.69\ V_{UBC}\ S_a$$

Table 2 presents the shear wall shear stresses in terms of the earthquake spectral acceleration.

The geotechnical consultant must provide the probability density function (PDF) of the random variable "50-Year Maximum Spectral Acceleration". This PDF must then be modified for the ductility level appropriate for the damage evaluation.

Our experience is that most geotechnical consultants will most probably provide Collapse and Damage Level response spectra. Such spectra use the definitions:

DAMAGE LEVEL RESPONSE SPECTRUM
This response spectrum corresponds to a 39 percent probability of not being exceeded during the life of the building. Unless otherwise specified, the life of the building shall be assumed to be 50 years.

COLLAPSE LEVEL RESPONSE SPECTRUM
This response spectrum corresponds to a 10 percent probability of not being exceeded during the life of the building. Unless otherwise specified, the life of the building shall be assumed to be 50 years.

Therefore, in effect these two "design spectra" give us two pieces of information about the value of the PDF at each natural period of vibration. In a two parametered PDF is selected to define the response spectra uncertainty for a specific natural period of vibration, then the two design spectra enable us to establish the values of these parameters for the specific building site. Figure 2 shows the PDF of the spectra acceleration that is assumed for this example that was obtained from a probabilistic response spectra and a ductility modification corresponding to the building natural period of vibration.

Using the PDF of the spectral acceleration and the results in Table 2, it is possible to calculate and plot the PDF of shear stress in any wall or pier. Let us concentrate our attention on the three piers of the wast wall. Figure 3 shows the PDF of the shear stress demand on piers 1 and 2.

The current code approach to unreinforced masonry shear wall structures is to specify an allowable shear stress capacity. The Long Beach ordinance for example, specifies a value

V_c = 4 psi (pre-field testing)

If tests are performed and non statistical methods are used, then the acceptable value of capacity is unclear. The confusion comes from the uncertainty in the results obtained using existing coring testing techniques. For example, consider actual data and a situation where nine 4" cores were taken in the field and the values, in psi, were (19.9, 19.8, 25.8, 7.0, 12.7, 35.4, 12.2 16.3, 12.9).

The test data can be used to obtain an estimate of the mean and standard deviation of the shear stress capacity. The values are:

1) mean shear stress capacity = 18.0 psi
2) standard deviation of shear stress capacity=8.04psi

Assuming that the shear stress capacity has a log-normal PDF, it follows that the PDF can be plotted as shown in Figure 4.

The three piers of the east wall are now analyzed to determine their reliability index values. A Monte Carlo analysis was performed using the following assumptions.

Shear Stress Demand
Type of PDF - extreme Type II
Form of PDF - see Figure 3

Shear Stress Capacity
Type of PDF - log normal
Form of PDF - Figure 4

Analysis Procedure
Monte Carlo Analysis
Number of samples - 500

Based on our Monte Carlo analysis, it follows that the reliability indices are:

PIER NO.	RELIABILITY INDEX	PROB. OF FAILURE
#1	0.8	22%
#2	0.5	35%
#3	0.7	0

The reliability index of the west wall was 1.0 and the north and south walls each had a value of 1.2.

Additional Monte Carlo analysis were also made to investigate the effects of a stronger wall material on reliability index and probability of failure. While doing these, the standard deviation of the stress capacity and the PDF is assumed to be the same as before, only the mean capacity increase to a larger value and shear stress demand is kept identical as before. Three mean shear stress capacities were assumed and the results are:

Set 1 Mean Shear Stress Capacity = 28 psi

Wall	Reliability Index	Prob. of Failure
Pier 1	1.8	0%
Pier 2	1.5	2%
Pier 3	2.8	0%
West	2.1	0%
North	2.3	0%
South	2.3	0%

Set 2 Mean Shear Stress Capacity = 36 psi

Wall	Reliability Index	Prob. of Failure
Pier 1	2.6	0%
Pier 2	2.3	0%
Pier 3	3.6	0%
West	2.9	0%
North	3.1	0%
South	3.1	0%

Set 3 Mean Shear Stress Capacity = 42 psi

Wall	Reliability Index	Prob. of Failure
Pier 1	3.3	0%
Pier 2	3.0	0%
Pier 3	4.3	0%
West	3.6	0%
North	3.8	0%
South	3.8	0%

Damage can be related to the value of the reliability index for one component, one failure mode, or alternately the average of a system of components. Table 3 recalls a previous definition of damage states. Figure 5 indicates a possible relationship between a damage state and a value of the reliability index. Therefore, for any calculated value of the reliability index, we can calculate a unique damage state. An improvement to this is to use the damage matrix concept as illustrated in Table 4. This approach incorporates uncertainty into the damage estimate for a calculate reliability index value of 3.0. Therefore, from the fourth column of the damage matrix, there is a 10% chance of damage state IV, 80% chance of damage state III, and a 10% chance of damage state II. This approach enables us to incorporate decision free concepts and evaluate potential dollar loss values for earthquake damage.

ACKNOWLEDGEMENT

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TABLE 1 OBJECTIVES AND CONTRIBUTIONS OF RELATED PARTICIPANTS

Participant	Objectives	Existing Capabilities Related to This Project
Geotechnical Consultant	Understand seismicity and earthquake wave propogational effects through the earth. Development of new methodologies which provide more accurate characterization of earthquake motions.	Can provide response spectra which incorporate the uncertainties in the characterization process. These spectra include uncertainties associated with the frequency of occurrence of earthquakes in the vicinity of the structure and the wave propogational aspects.
Code Developers and System Reliability Analysists	Understand the uncertainties inherent in the many aspects of the design/analysis process. Development of new methodologies which provide more accurate characterization of structural component and system reliability. Establish acceptable levels of component and system reliabilities for different limit states (i.e. failure modes). Present the results of their studies in a code format that can be used by engineers who do not have the technical expertise to understand the mechanics reliability analysis.	Can provide analysis techniques for calculation of system reliability. Can provide insight into uncertainties associated with material properties analysis formulas versus component performance and structural load levels, particularly live and dead loads. Have quantified recommended levels of reliability.
Experimentalists and System Identification Analysists	Development of improved techniques to test real structures in order to determine material and system properties.	Testing laboratories can determine material properties using coring, material coupon or similar structural specimens. System dynamic response parameters can be quantified at ambient response levels (e.g. natural frequencies and mode shapes).
Structural Dynamicists and Computer Consultants	Development of computer structural modeling techniques. These techniques enable the development of more refined analytical models and seek to reduce computational efforts.	Computer programs exist for detailed linear and non-linear building analysis. Micro-computers are available along with a large number of computer programs.

TABLE 2 SHEAR WALL SHEAR STRESSES (PSI)

	CODE	SPECTRAL
North Wall	8.2	22.1 S_a
South Wall	8.2	22.1 S_a
West Wall	10.2	27.4 S_a
East Wall - Pier #1	12.9	34.7 S_a
East Wall - Pier #2	16.8	43.7 S_a
East Wall - Pier #3	2.7	7.3 S_a

S_a = spectral acceleration (g's)

TABLE 3 DEFINITION OF DAMAGE STATES

Damage States

(1)	Minimal Damage:	No appreciable financial loss.
(2)	Minor Damage*:	Can be repaired with small interference with normal operations.
(3)	Moderate Damage:	Can be repaired with small interference with normal operations; perhaps equivalent to closing down for several days.
(4)	Severe Damage:	Significant damage to structural members; repairs will require closing for at least several weeks.
(5)	Major Damage:	Extensive damage to structural elements; repairs require closing down for several months.
(6)	Partial Collapse:	Repairs require closing down for an extended period, from 5 months to one year.

*Damage states (2) to (6) after Housner and Jennings

TABLE 4 DAMAGE MATRIX

	Reliability Index					
Damage State	$\beta \leq 0$	$\beta=1$	$\beta=2$	$\beta=3$	$\beta=4$	$\beta=5$
VI	0.90	0.10				
V	0.10	0.80	0.10			
IV		0.10	0.80	0.10		
III			0.10	0.80	0.10	
II				0.10	0.80	0.10
I					0.10	0.90

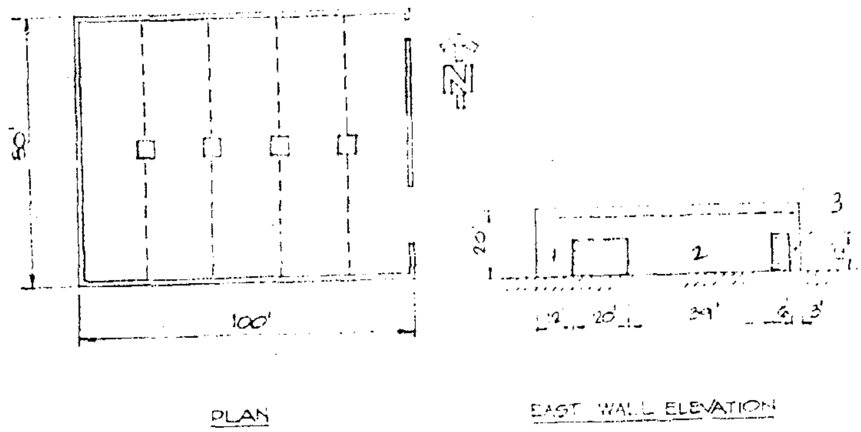

Figure 1   Plan and Elevation of Illustrative Building

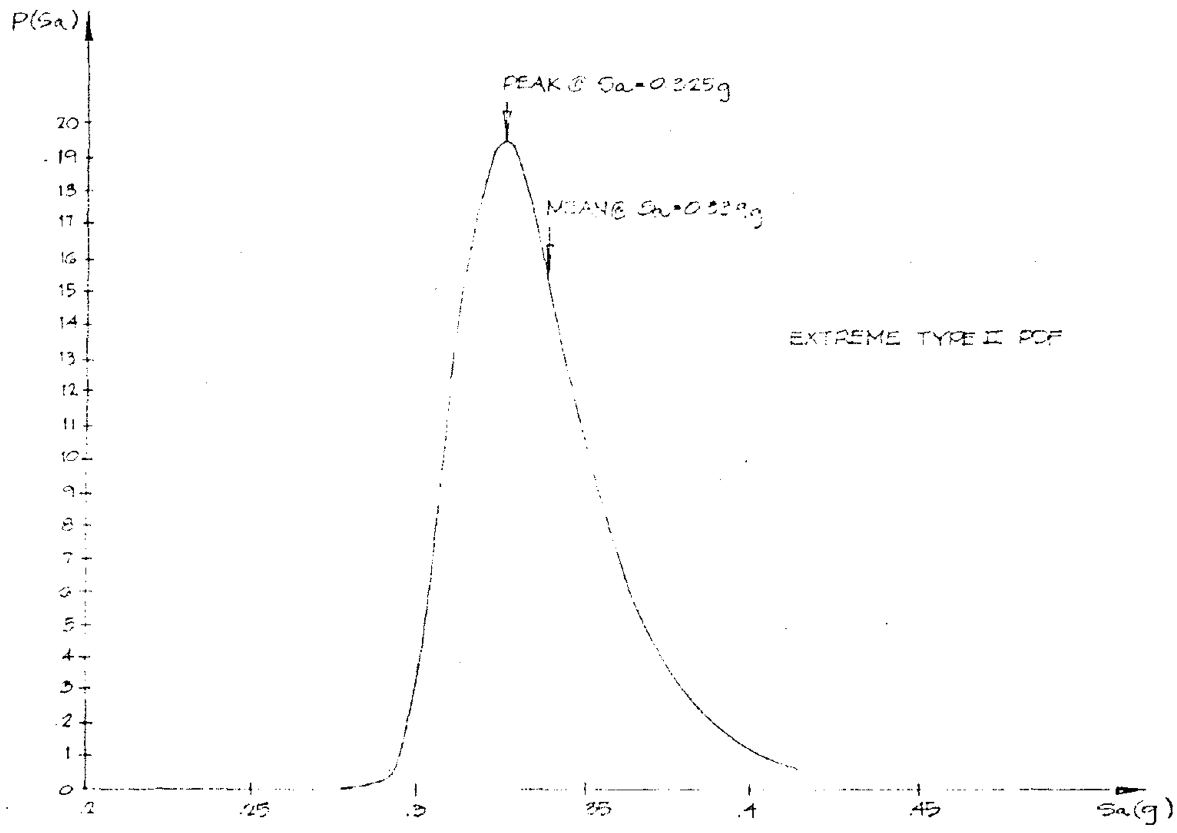

Figure 2   PROBABILITY DENSITY FUNCTION OF THE RANDOM VARIABLE S_a = 50 YEAR MAXIMUM INELASTIC SPECTRAL ACCELERATION

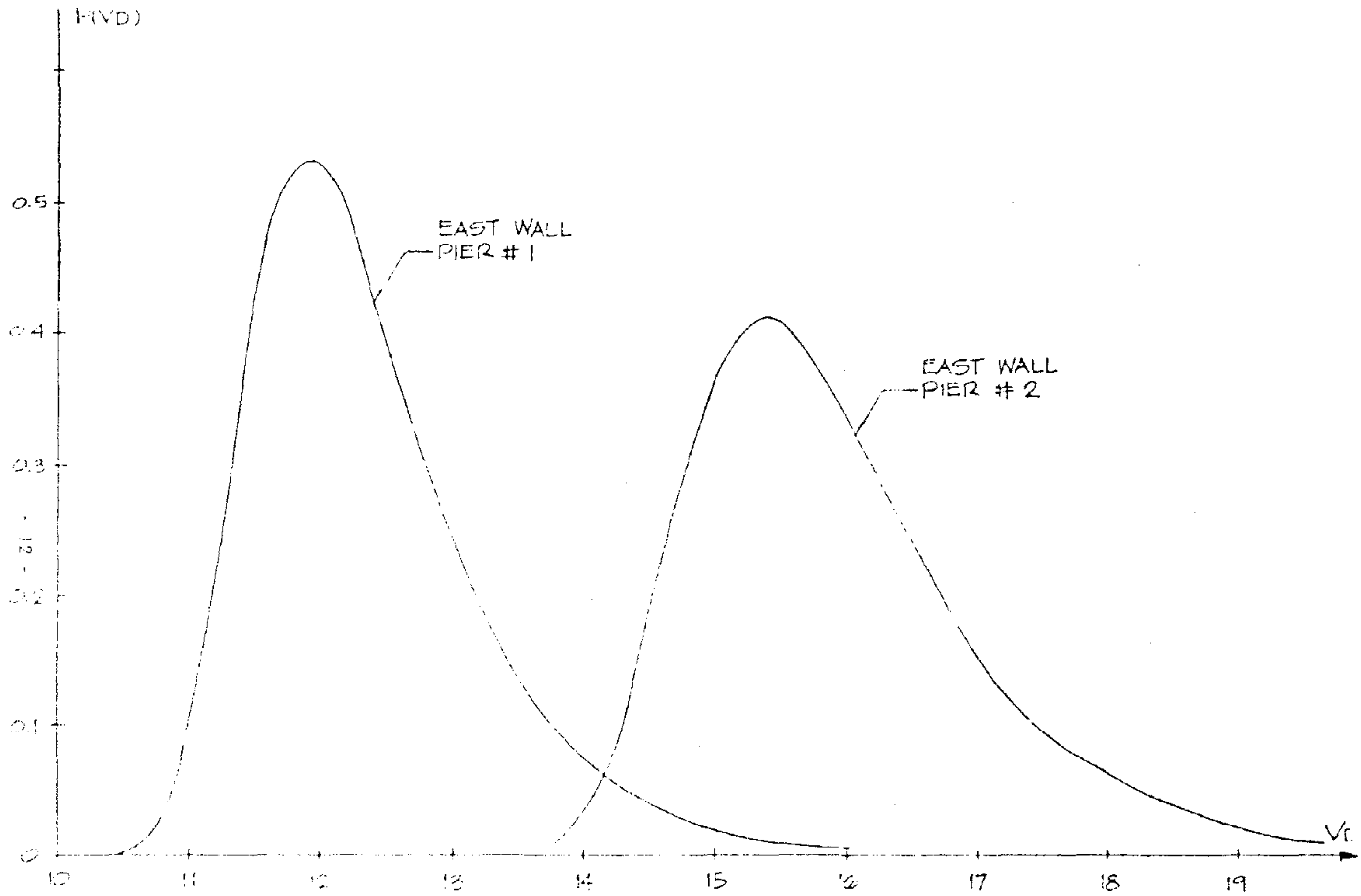

Figure 3 SHEAR STRESS DEMAND

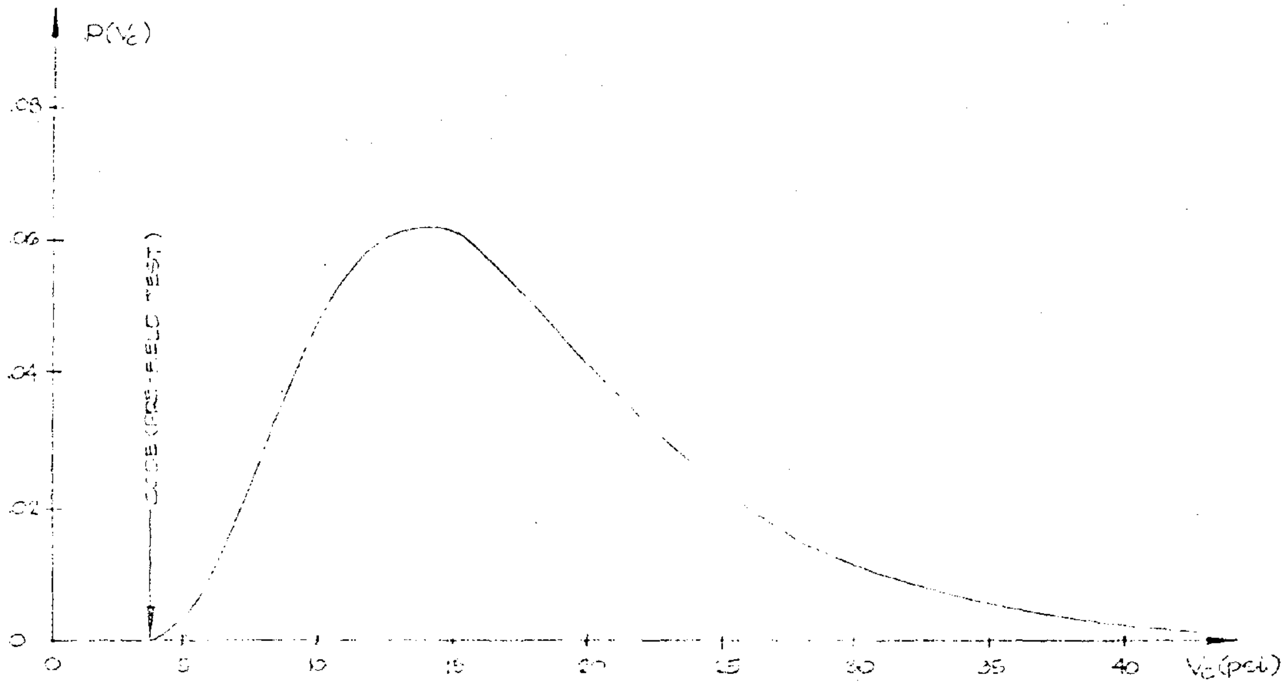

Figure 4 PDF OF SHEAR STRESS CAPACITY

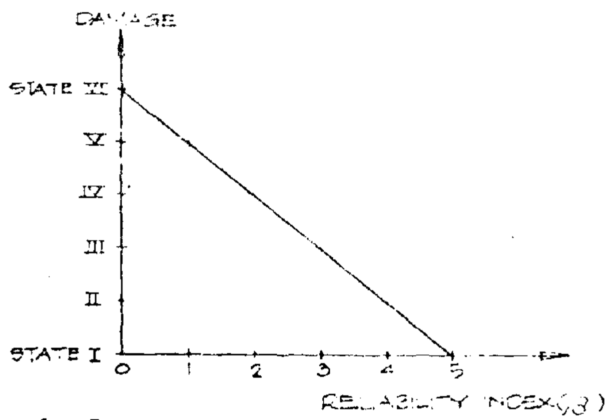

Figure 5 Damage verses Reliability Index

ASSESSMENT OF REINFORCED CONCRETE BRIDGE SLABS

R. J. Cope, J. H. Bungey, P. V. Rao

Department of Civil Engineering, University of Liverpool, England.

SUMMARY

The integrity of a reinforced concrete bridge slab may need to be assessed when the structure is showing signs of possible distress, excessive cracking or deflections, when an overload is expected; and when structural modifications are desired. Methods of estimating concrete strength in prototype structures are described, and interpretation of such measurements for use in calculations is discussed. Use of non-linear finite element procedures to predict the complex stress state compatible with measured deflections is described. Methods for extending the analysis to estimate the effects of overloading and structural modifications are proposed.

INTRODUCTION

Assessment of a reinforced concrete bridge slab is required when there is visual indication of distress, when a particularly heavy loading is to be carried, and when structural modifications are contemplated.

Bridge slabs built to current standards are unlikely to exhibit excessive cracking and displacement due to gravity loading, but some skewed and curved structures that were designed before there was a proper understanding of the factors affecting their stiffness and strength have given rise to concern (see, for example, New Civil Engineer, 1981).

Some top slabs of integral beam and slab bridges have shown distress that has led to spalling and corrosion of steel (see, for example, Maeda et al, 1980; Csagoly et al, 1980). Interestingly, assessment of such structures by field tests has indicated that response to concentrated loading is closer to that predicted by non-linear analysis, than by the linear plate theory commonly used in design.

Assessment for overload conditions is likely to increase in importance as there is pressure in many countries to permit heavier axle loads and increased traffic flow is an almost universal phenomenon.

Field testing can be broadly classified under material testing and load testing for structural response. Load testing is immensely valuable for assessing overall structural behaviour and for assessing assumptions made in design. However, as it provides no detailed information on material properties and is expensive, it tends to be used only as a last resort. In this paper, attention is concentrated on cheaper and less disruptive methods of non-destructive testing for assessing concrete strengths. The likely variations of material properties over plan and depth of a bridge structure are described. The relationship between site measurements, equivalent standard specimen and design strengths are discussed.

For evaluating the response of existing structures, and for better understanding of structural behaviour, in addition to field testing, relevant analytical methods are needed. Since non-linear finite element techniques have the capability to utilise data from 'as built' drawings and non-destructive testing they seem to offer a suitable method for development. Once the finite element material model is properly calibrated, its predictive ability can be used to check alternative modes of alleviating distress and response to overloading. By providing insight into structural behaviour they could also lead to improved design procedures.

IN-SITU MEASUREMENT OF CONCRETE STRENGTH

In-situ measurement of concrete strength for use in calculations is not an easy task. The difficulties arise in three ways:

(a) Practical application of test methods.
(b) Interpretation of results in the light of test calibrations and in-situ strength variability.
(c) Interpretation of results for use in calculations.

Although these factors are inter-related, it is convenient to discuss them under separate headings.

Practical Application of Test Methods

The various non-destructive tests which are available have been widely described in technical literature. The principal methods of use for slabs are Rebound Hammer, Ultrasonic Pulse Velocity, Windsor Probe, Internal Fracture and Cores. Table I indicates the principal features of these methods, relative to each other, assuming good access and test conditions.

It has become apparent, however, from investigations of a number of highway bridge deck slabs that good test conditions are seldom available. Top surfaces generally have surfacing which must be removed if tests involve access to this surface, and even after removal the remaining concrete surface is seldom in a suitable condition for application of rebound hammer or ultrasonic pulse velocity methods without preparatory work such as grinding down. This negates the chief advantages of these methods which are speed and low cost enabling a comprehensive survey involving many readings. It has also been found that where surfacing patches are removed these areas trap surface water and hence give moisture conditions which are unrepresentative of the deck as a whole, and can cause calibration problems for rebound hammer and ultrasonic methods. Although rebound hammer readings could be taken on the soffit, which usually has a reasonably smooth surface, these should not be used for strength estimation where the concrete is more than three months old due to the possibility of carbonation effects.

Method	Cost	Time	Damage	Zone Assessed	Reliability of Strength Calibration
Rebound Hammer	Low	Fast	None	Surface	Poor
Ultra-sonic	Low	Fast	None	Good Average	Mod.
Windsor Probe	Mod.	Fast	Minor	Near Surface	Mod.
Pull-out	Mod.	Fast	Minor	Near Surface	Mod.
Cores	High	Slow	Mod.	Any	Good

(Mod. ≡ Moderate)

Table I - Relative Features of In-situ Test Methods

It is seldom practicable to take ultrasonic pulse velocity readings on the soffit only due to flexural and shrinkage cracks which are usually present. Although direct measurements through the depth of the member may be possible with suitable top surface preparation, it has been found that reliable results cannot be obtained in regions of visible cracking. This is due to the effects of internal cracks on both path length and width. In regions where reliable readings are possible it is essential that absolute strength estimation is based on a calibration for the particular mix.

Windsor probe and pull-out (or internal fracture) tests do not require the same surface preparation, but surfacing must be removed. Both tests relate to concrete near the surface, and leave localised damage to be made good. Whilst pull-out tests can yield an estimate of strength on the basis of a generalised calibration curve the accuracy is poor. Windsor probe testing requires a calibration for the appropriate aggregate type and hardness, and provided this is available, is an easier and more reliable method to perform under site conditions. The method has been successfully used by the Authors on bridge decks whilst in service, and a test rate of four locations per hour has been found reasonable for one operator working on a deck soffit using a small mobile platform. Readings on a prepared top surface could be made at up to twice that rate.

Cores offer the most reliable method of concrete strength determination, but are expensive, slow and often difficult to obtain. Consequently, the number of test locations will be limited since groups of at least 3 cores are required at each location to obtain a reliable strength estimate. Special care must be taken to follow the recommendations of an appropriate 'standard' concerning cutting, capping and testing. This will normally require a minimum diameter of 100mm, which should be possible with most slabs and with care, an estimate of strength at a particular depth of slab may be possible. The temptation to economise by using smaller diameter specimens should be resisted, since the accuracy will be considerably reduced.

A combination of Windsor probe and core testing has been found to have advantages when a limited number of cores can be used to confirm calibration of the Windsor probe which is used for a more widespread survey. For both methods, reinforcement should be located by covermeter and avoided if possible.

Interpretation of Results

Firstly, the results of individual tests must be assessed to provide strength estimates at particular locations, together with the likely accuracy of those estimates. When that has been done, the variability of estimated strengths over the slab can be assessed, and a view formed of the representativeness of the results in relation to the whole deck.

Under ideal conditions of test and availability of calibrations the accuracies of strength prediction at a particular in-situ location are unlikely to exceed the values given in Table II (Bungey, 1981). As mentioned above, ideal conditions seldom exist for prototype structures, and estimates are, therefore, likely to be of even lower accuracies.

Method	Maximum Likely Strength Accuracy (95% Confidence Limit)
Ultrasonic	±20%
Windsor Probe	±20%
Pull-out	±30%
Cores - large ≥100mm	± 6%
(4) - small	±18%

Table II - Strength Accuracies Under Ideal Conditions

It is well established that considerable variations in in-situ concrete strength are to be anticipated due to compaction, curing and material supply variability. Variations in plan for deck slabs are primarily likely to be due to supply and compaction effects and are, therefore, of a random nature. A coefficient of variation of 15-20% on concrete strength has been suggested for average quality construction (Mirza, 1979), whilst for excellent construction standards a 10% coefficient of variation is to be expected for typical structural quality concrete. These figures are based on site made, laboratory cured control specimens, with a low testing variability. Greater variations may be expected for the same concrete on site.

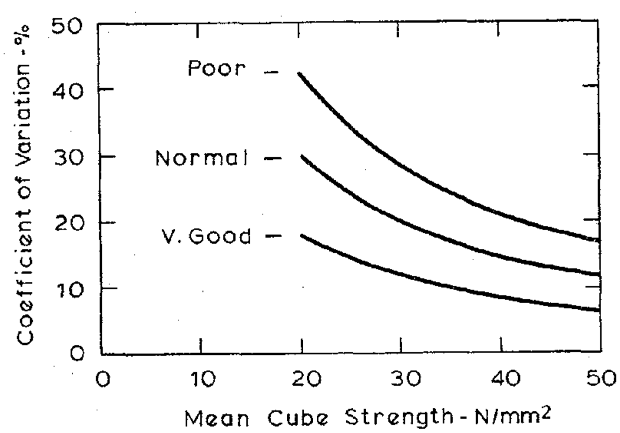

Fig. 1 -- Variability of In-situ Concrete Strength measurements relative to construction quality

It is well known that for a particular level of construction quality, the coefficient of variation is a function of the mean concrete strength. Figure 1 suggests likely values of coefficient of variation for mean in-situ strength measurements relating to three levels of construction quality based on a variety of European and North American sources. From this figure it can be deduced that for a typical bridge concrete of $40N/mm^2$ mean strength, for example, a standard deviation of $6N/mm^2$ is likely for normal quality construction. This corresponds to 95% confidence limits of $\pm 10N/mm^2$ about the mean value. The use of strength estimates based on mean in-situ test values must make due allowance for this, as well as for the accuracy of the test itself.

Variations in strength through depth of well compacted bridge slabs are predominately due to curing effects. It has been demonstrated that for 500mm. deep beams a considerable strength reduction may be expected to exist from bottom to top, even when construction is under laboratory conditions (Bungey, 1980). Similar strength differentials are anticipated in slabs of this and greater thicknesses. The published evidence available for thin slabs of 100-200mm. thickness, however, suggests a smaller strength reduction which is concentrated near to the top surface. These features are summarised in Figure 2. A further aspect with thin slabs is that trowelling of the surface can produce a hardening of the surface layer and this can affect some test results unduly.

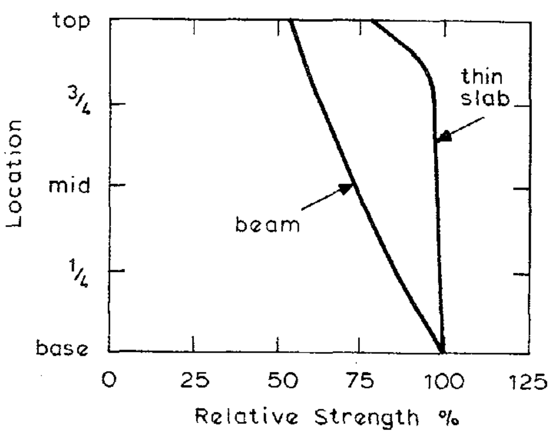

Fig. 2 -- Relative Strength Distribution Through Thickness

Field tests by the authors on a deep voided skew highway bridge deck slab which was approximately ten years old were carried out by the Windsor Probe method. Readings were taken at corresponding locations on the top surface and soffit and indicated minimum top surface strengths of 72% soffit values, but with a mean of 88%.

Strength variations with depth must thus be carefully considered when planning the location and type of in-situ testing. Surface zone tests will indicate the extreme values, but cores may not detect localised surface features and ultrasonic measurements across the depth of slabs will yield an average value. Where values are required for calculation purposes it is essential that they should relate to the critical regions of the member. If testing cannot produce this directly due to practical difficulties, the value must be estimated bearing in mind the features indicated in Figure 2. Where results are to be used for comparison with code specifications, however, it is essential that they reflect an average value of strength across the member.

Modulus of Elasticity

Whilst long-term deformation is dominated by creep and cracking, analysis of live load influences requires an estimate of the short-term elastic modulus (E_{sh}). This can be deduced from measurement of strength or dynamic modulus. Since elastic modulus is dependent both on aggregate type and proportions, the relationship to concrete strength is not clearly defined. The problems of strength estimation have been described, and it is unlikely that a value of elastic modulus could be deduced to an accuracy of better than ±15% even for an accurately known concrete strength. If a more precise value of E_{sh} is required, a value of Dynamic Modulus can be obtained from Ultrasonic pulse velocity results and then adjusted to give a corresponding value of static modulus for use in calculations.

Use of Measured Material Properties in Calculations

Design calculations use properties related to the characteristic strength of standard specimens with factors of safety. For non-linear analysis, the Model Code (CEB-FIP, 1978) recommends a stress-strain curve for loads of short duration as a function of the characteristic specimen strength and the associated value for E_{sh}. For an existing structure the relationships between in-situ and standard specimen strengths, between characteristic and mean strength, and the use of appropriate factors of safety have, therefore, to be considered.

In-situ strength may be less than that of a laboratory cured standard specimen of the same concrete due to differences in compaction and curing. The extent of this difference depends on many practical factors, but for a wet in-situ concrete in a thin slab the average 28-day equivalent cylinder strength may be as low as 65% of that of a standard specimen. For a deep slab or beam the corresponding figure would be about 90%. For a dry in-situ concrete, such as might occur under the waterproof membrane of a bridge slab, the strength may be 10% higher than for a wet concrete. However, when estimating likely future concrete strength, long-term increased strength development should not be relied upon, since it is subject to factors such as cement type, curing and environment.

Field measurements produce an estimate of the mean strength of the in-situ concrete $\bar{f}_c$ for the locations tested. The characteristic, or lowest acceptable strength, f_c, is normally assumed to be related to the mean strength by the relationship:

$$f_c = \bar{f}_c - 1.64s \qquad (1)$$

where s is the standard deviation. When insufficient test results are available to enable a direct statistical analysis, the value of s may be derived from the data presented in Figure 1, provided that testing has been concentrated on areas within the structure which are likely to give comparable strengths (e.g., top or soffit of slabs or beams). The values in Figure 1 do not encompass the strength variations with depth suggested by Figure 2, and if readings have been taken at locations likely to include such variations it will be necessary for the engineer to use his judgement to arrive at a suitable estimate of standard deviation s for use in calculations.

The value of in-situ f_c thus obtained is used for non-linear analysis and, in the light of the above comments, is likely to be a safe-side estimate. If an average E_{sh} has been obtained from testing it seems reasonable that this should be used directly for analysis. Otherwise, a value appropriate to a concrete with characteristic strength f_c should be taken from the literature.

In normal design procedures for determining the strength of sections concrete parameters are derived from the characteristic specimen cube or cylinder strength. Partial factors of safety are introduced to cover a variety of features including the difference between in-situ and control specimen strengths. Provided that the in-situ tests are located at critical regions considered in calculations, it seems reasonable to dispense with the factor which compensates for this difference. There will, however, always be uncertainties about the accuracy of test data and possible future deterioration of a structure and it is prudent to retain a factor of safety. A minimum design value of (f_c/1.2) is recommended.

NON-LINEAR FINITE ELEMENT ANALYSIS

The finite element method is too well known to be detailed here. The slab studies presented are based on a thin plate formulation (Baldwin, 1973). Stiffnesses of steel and concrete are obtained separately, with composite action being achieved through the assumption of perfect bond (Cope, Rao, 1977). Steel and concrete stresses are determined from the prevailing strain field, and assumed constitutive equations, at a grid of sampling stations over each element. In plan, these stations are located at the 2x2 Gauss integration points used to determine the element stiffness matrix and the 'released' nodal forces due to material degradation. Reinforcement is 'lumped' at these stations. The depths and directions of the outermost steel layers in each direction close to the soffit and top surface are retained, but for economy inner layers are grouped at their respective centres of gravity. Concrete stresses are determined at five equally spaced stations through the slab thickness and the appropriate Newton-Cotes formulae are used to determine integrals for in-plane forces and moments about axes in the median plane.

Loading is applied in increments and the tangent stiffness matrix is held constant over each load increment. Stiffness degradation due to material damage is simulated by comparing internally generated forces with applied loading. An iterative procedure is followed until an equilibrium position is reached to some prescribed tolerance (Cope, Rao, 1981). Using suitable constitutive equations, it is possible with this method to follow the complete load history of a slab.

For assessment of a precracked prototype structure that has been subjected to an unknown load history constitutive equations can only be approximated. It has been shown (Sparks, 1973) that for beams, the effects of fluctuating load with a sustained component are similar to those of long-term application of the loading at its maximum intensity. Although, to the authors' knowledge there is no comparable evidence for slabs, where the crack directions may be dependent on load history, it is recommended that this result be used to enable assessments to be made from analysis of a single load case.

It is proposed that the behaviour of a slab be estimated from a two-stage analysis. In the first stage, the effects of sustained and repeatedly applied loads are evaluated. The resulting model is then used as the basis for calculating response to abnormal loading and effects of structural modifications. Material models appropriate to the two phases are discussed next.

Material Property Model

Constitutive equations are based on simplified material models. These have to take into account such diverse phenomena as creep, shrinkage, bond-slip and debris in cracks.

For reinforcement, identical tri-linear uni-axial stress-strain curves are used in tension and compression. Different types of reinforcement may be used in a bridge slab and properties appropriate to each should be specified. As standards relating to steel properties evolve with time, documents contemporary with the construction period should be studied when more detailed evidence is not available.

To initiate the analysis for long-term loading, stiffness of concrete is calculated based on unstressed properties. These are, for plane stress conditions, assuming isotropy:

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{Bmatrix} = \frac{E}{(1-\nu^2)} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_{xy} \end{Bmatrix} \qquad (2)$$

where $E = E_\ell$, estimated long-term Young's modulus; and
ν = Poisson's Ratio.

Forces mobilised by concrete are calculated using idealised, uniaxial stress-effective strain relationships (3) in principal directions.

$$\begin{Bmatrix} \varepsilon_1^* \\ \varepsilon_2^* \end{Bmatrix} = \frac{1}{(1-\nu^2)} \begin{bmatrix} 1 & \nu \\ \nu & 1 \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \end{Bmatrix} \qquad (3a)$$

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \end{Bmatrix} = \begin{bmatrix} E & 0 \\ 0 & E \end{bmatrix} \begin{Bmatrix} \varepsilon_1^* \\ \varepsilon_2^* \end{Bmatrix} \qquad (3b)$$

Concrete cracks when principal tensile stress exceeds tensile strength. It has been shown (Cope, Rao, 1980) that analytical response of slabs monotonically loaded to failure under laboratory conditions is sensitive to values of tensile strength and representation of tension stiffening. However, for slabs subjected to loading with a significant sustained component and repeated applications of additional multiple load patterns, there is evidence to show that tensile strength and tension stiffening are much less significant (Cope, Rao, 1981). Tension stiffening can, therefore, be neglected for current analyses, and the stress component orthogonal to cracks is set to zero. Poisson's Ratio is set to zero at a cracked station.

When stresses are evaluated in principal directions, concrete shear modulus is not used. The principal directions can be viewed as defining material property axes, with cracks crossing orthogonally to principal strain direction. Principal directions can change with redistribution induced by cracking. It is not suggested that actual crack directions rotate, but that cracks can form in more than one direction. Local concrete stiffness is most influenced by the crack whose direction is closest to the axis of principal moment for each particular load pattern. This model is clearly an approximation to actual material behaviour, but has been shown to give good predictions of short-term structural response (Cope, Rao, 1981).

Analytical Procedure

To determine the state of strain in a cracked prototype structure it is analysed with E set to an appropriate long-term value of Young's Modulus. The loading applied is the dead load plus serviceability loading applied as a u.d.l. Crack positions and inclinations are recorded and the live load component is then removed.

When equilibrium between internally mobilised forces and the permanent loading is established, the effects of abnormal loading and structural modifications can be assessed. The constitutive equations (2-3) are used, but with E set to E_{sh}, the short-term value for Young's Modulus. The uni-axial stress-strain curves used to determine additional internal mobilised forces in cracked and uncracked directions are shown in Figure 3. It can be seen that cracking is permitted in the presence of an overall compressive strain. This is usual when part of the total strain is due to time related effects. Also, a compressive stress can be carried whilst there is a total tensile strain. This is to account for the effects of bond slip and debris in cracks. At present there is insufficient experimental evidence available to thoroughly check these models, but the general approach is in accord with observed behaviour.

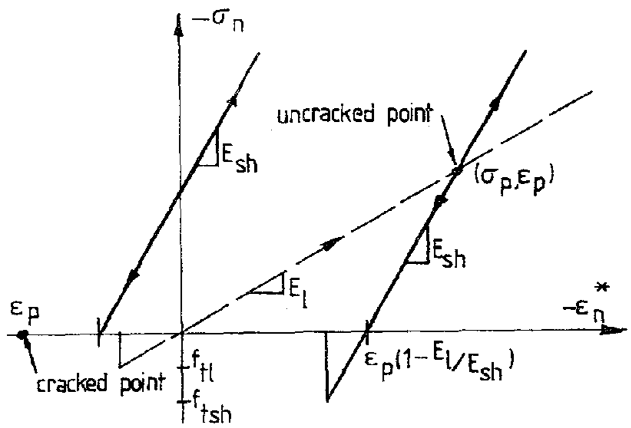

Fig. 3 -- Stress-effective Strain Curves for Long and Short-term Loading

When the effects of structural modifications, such as the addition of intermediate supports, are to be studied, the tangential stiffness matrix is modified by incorporating the support stiffness. In some cases, jacking may be used at a newly introduced support to enable it to carry part of the dead loading. This may be the case when existing bearings are overloaded or original shear stresses are dangerously high. As long-term effects are involved, a new set of values for (σ_p, ε_p) are determined. This is done by imposing prescribed forces or displacements at the jacking points in accordance with the practice to be used on site.

An alternative approach for dealing with time effects has been proposed for beams (Scordellis, 1981) in which a further iterative cycle is introduced to allow for creep effects. This may prove to be a better approach for structures with a known load history, but to date there is insufficient experimental data to justify the additional analytical costs.

With either approach, specification of material properties is subjective and an engineer can set them to give predictions that model observable features such as deflections and cracking of prototype structures.

Material Properties to be Used for Analysis

As the values given to E_ℓ, $f_{t\ell}$, ν are necessarily subjective, it is interesting to examine their effects on predictions of behaviour of a deep prototype skew bridge slab for which self weight was the major loading. In Figure 4a the extent and directions of soffit cracking predicted for long-term loading with $E_\ell = E_{sh}/3$, $f_{t\ell} = 0.08f_c$ and $\nu = 0.2$ are shown. Analyses with E_ℓ set to values in the range $E_{sh}/3 \leq E_\ell \leq E_{sh}$ produced very similar results. Typical mid-span deflections are given in Table III. It can be seen that, with a non-linear analysis, the value of E_ℓ selected affects both the average and distribution of deflections.

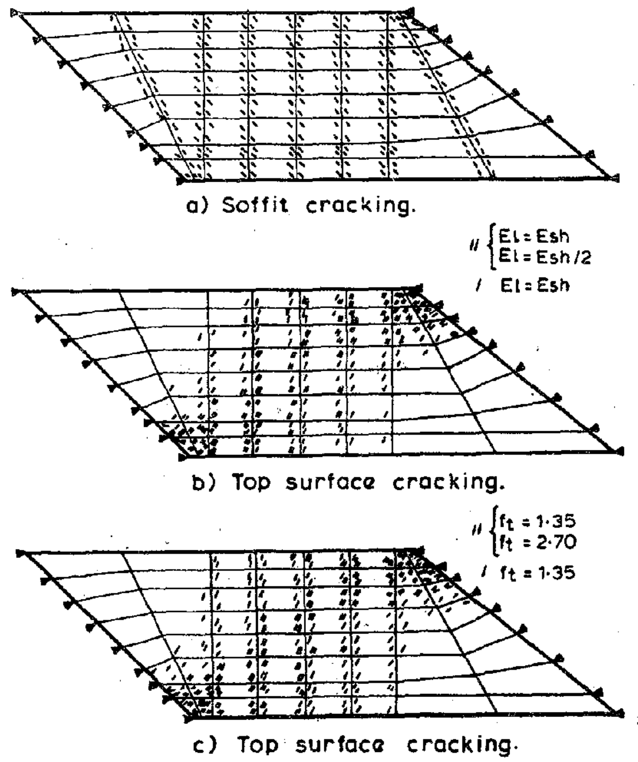

Fig. 4 -- Comparison of Predicted Crack Patterns

In Figure 4b the predicted top surface cracking for $E_\ell = E_{sh}$ is compared with that for $E_\ell = 0.5E_{sh}$. For both analyses $f_{t\ell} = 0.08f_c$ and $\nu = 0.2$. It can be seen that the spread of cracking is greater with $E_\ell = E_{sh}$. As the permanent loads are 'applied' over a short time interval (e.g., when the shutters are struck) the predictions with $E_\ell = E_{sh}$ are relevant. It is not reasonable to suppose that the cracks heal with the passing of time. To produce a crack pattern similar to that obtained with $E_\ell = E_{sh}$, but with average deflection appropriate to long-term effects from a single analysis, $f_{t\ell}$ was set to $0.04f_c$ and $E_\ell = 0.5E_{sh}$. (For this analysis the strain at which cracking is initiated is the same as for the analysis with $E_\ell = E_{sh}$ and $f_{t\ell} = 0.08f_c$).

In Figure 4c the crack patterns predicted for $f_{t\ell} = 0.08f_c$ and $f_{t\ell} = 0.04f_c$, with $E_\ell = 0.5E_{sh}$ and $\nu = 0.2$, are compared. The mid-span deflections are given in Table 3. It can be seen that the central deflections are similar. Reducing the tensile strength increases the transverse hogging curvature which leads to the desired greater spread of top surface cracking.

E_ℓ/E_{sh}	$f_{t\ell}/f_c$	Edge	Centre	Edge
1.0	0.08	129	119	161
0.75	0.08	136	127	170
0.5	0.08	149	141	185
0.33	0.08	166	161	207
0.5	0.04	161	146	202

Table III - Mid-span Deflections (mm)

Analyses performed with Poisson's Ratio in the range 0.1 to 0.2 indicated that predicted behaviour was not sensitive to the value of this parameter. Cover to reinforcement was decreased by 10% and increased by 30%, values which were considered to be extremes of allowable tolerances, but predicted behaviour was not significantly affected. Provided as-built drawings are reasonably accurate, the major parameters to be set are E_ℓ and $f_{t\ell}$. The range of values discussed above have been found to give results reasonably in accord with observed behaviour of a prototype bridge slab in service and of laboratory tests (Cope, Rao, 1981). More data on bridge behaviour has to be gathered before general recommendations on material property values can be made.

CONCLUSIONS

Non-linear finite element methods can be used to estimate the flexural response of bridge slabs in service to overload and structural modification. The material data needed for analysis of short-term effects can be obtained by non-destructive tests. Because of variation in properties, and the uncertainties involved in their measurement on site, simple material models are recommended for use in analysis.

Studies using non-destructive testing show a random variation of concrete properties in plan and an increase in strength through thickness, though possibly with a thin layer of 'work-hardened' concrete on top. At present, parameters for the analysis of long-term effects cannot be obtained objectively, and estimates may need to be adjusted to ensure that predicted deflections and crack patterns model those observed. In view of the uncertainties, it is recommended that characteristic values of properties be used at all sampling stations.

It would be useful if authorities kept a record of bridge displacements to assist with assessments.

ACKNOWLEDGEMENTS

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BRIDGE EVALUATION BY FIELD TESTING

C. P. Heins,
Professor, Institute for Physical Science and Technology
Civil Engineering Department
University of Maryland
College Park, MD 20742 USA

SUMMARY

The cost of replacement of many old truss bridges is often prohibitive, however such bridges provide a very necessary service to the community. Thus the integrity of these bridges is paramount, in order to insure safety for the public.

This paper will therefore present information relative to the load carrying capacity of six truss bridges, as observed during load field tests. The resulting data were then used to rate the structures.

INTRODUCTION

1. General

Throughout Frederick County, Maryland, and possibly many other counties in Maryland, there are many steel truss bridges which were built in late 1800. Most of these truss bridges are on secondary roads, and carry a single lane of local traffic. Their importance, however, cannot be minimized, as they supply a great need to the local community.

All of these bridges have been inspected and subsequently rated. In many instances the ratings are so low that only passenger cars are permitted to cross the bridge. Thus school busses, farm equipment and trucks must find an alternate route or the bridge must be replaced or repaired, to up grade its capacity. The cost of replacing these bridges would be prohibitive, therefore the solution is to repair and strengthen the bridges. The actual strength of the bridges, and thus a reliable rating, can only be assured if the bridge is examined under a typical truck loading.

This paper therefore will present the results of tests on six such steel truss bridges and the effects that strengthening of the deck had on the floor system. Also to be presented are the results of laboratory tests on stringer beams, when the beams are stiffened with nailers and planking. The results yield a distribution factor which is less conservative than AASHTO.

2. Problem

The useful carrying capacity of bridge elements can be determined by analytical methods or experimental tests. When the structural system is highly indeterminant, the simplified analytical methods can greatly underestimate the load carrying capacity. In the instance of bridge structures, which are highly indeterminant systems, the ultimate load capacity may be fifteen times the elastic capacity.

Six truss bridges throughout Frederick County, Maryland, have been examined and the allowable load capacity determined by simplified analytical techniques. These results recommend a severe lowering of the carrying capacity of the bridges. These results may be reasonable at face value. However, from field observations it has been noted that the bridges have supported much greater loads without any noticeable detrimental effects to the structure. Therefore, there is some disagreement between the analysis results and what is actually occurring in the structure under load.

This discrepancy can be overcome by testing six selected bridges and certain components in the laboratory under a known load and proper instrumentation. The results of such testing and thus rating of the bridges will be presented herein, and is the objective of this paper.

3. Program Details

The following paragraph will describe the program details that were required to meet the objectives of this study;

1. Select six (6) bridges which have been inventoried, inspected, and rated for test purposes.
2. Examine and study the "Bridge Inspection and Rating Reports", for each bridge and locate critical members.
3. Determine the location of strain gages for each bridge.
4. Determine the field truck loading position.
5. Instrument bridge with the strain gages.
6. Test bridge (i.e., position loaded truck at various locations on the bridge and read gages).
7. Reduce data and develop rating.
8. Test typical floor beams in laboratory with/ without nailers.
9. Compare field and laboratory tests results with analytical data.
10. Prepare recommendations for the rating of the bridges.

BRIDGE FIELD TEST STUDY

1. Bridge Description and Gage Locations

The six truss bridges that were field tested consist of single lane steel pinned jointed truss structures approximately 15.0' wide and spans of 60' to 90' long. With the exception of Gapland Road bridge, the other five bridges have wooden decks. These decks are nailed to nailing strips which are attached to the steel stringers. The deck members are generally wooden 3" x 10" and the nailers are wooden 3" x 10" either of pine or oak. The Gapland Road bridge deck consists of a steel grating.

Each of the six bridges was instrumented with SR-4 strain gages at various positions.

2. Loading

The truck load that was applied to each bridge consisted of a gravel loaded 2D truck. The rear axle weight is of the order of 22.5 kips, which was positioned along the bridge at various locations, as dictated by the influence lines, to institute maximum effects. These trucks are similar to the design H-15 vehicle, which has a rear axle weight of 24^K and front axle weight of 6^K. This type of vehicle represents the worst possible load to which each bridge might be susceptable.

3. Test Results

The test truck was placed on each truss bridge at various positions, and the induced strains on the various members recorded. In some instances, the strain gages were monitored for both static and dynamic loading.

The resulting induced stresses are given in Tables 1 through 6 for each of the six bridges. The gage number identification refers to the gage locations on the bridge. The tabulated stresses were obtained by converting the strain readings from the elastic modulus of 30 x 10^3 ksi, as obtained from coupon tests.

BRIDGE DESIGN STUDY

1. Bridge Rating

As mentioned previously, each of the six bridges has been rated, as based on a complete analytical study. The resulting ratings were obtained after computing the probable induced dead and live load stresses and the allowable stresses. The allowable stresses were selected from the AASHTO Specification in "Manual for Maintenance Inspection of Bridges" and field observations, i.e., a reduction in the design value of certain members was used if the member was noted to be distorted or corroded. With these criteria and field observations of the various members, the rating of each bridge was computed. The ratings correspond to H-15 truck with an appropriate reduction.

Those members which govern the rating of each bridge are as follows:

Longs Mill Road	Stringers	Gages 4, 5	H0
Crum Road	Floor Beam	Gage 2	H2
Water Street	Diagonal	Gage 1	H1.2
Daysville Road	Stringer, Floor Beam	Gages 1, 5	H3
Old Hagerstown Road	Stringer	Gages 3, 4, 5	H1
Gapland Road	Diagonal	Gage 7	H5

Assuming the dead load stresses as computed are accurate, it remains to establish the accuracy of the live load stresses. Such stresses have been obtained through tests, described herein on Tables 1 through 6. These field live load stresses, in conjunction with the computed dead load stresses, have been used to re-evaluate the bridge ratings.

LABORATORY STUDY

1. Purpose of Tests

As described in the previous section, "Design Study," three out of the six bridges have ratings which are governed by the strength of the stringers. In most instances, these low ratings are due to the conditions that no lateral bracing of the compression flange is assumed. In order to rectify this condition, the County Engineer has devised a method of providing continuous lateral bracing by attaching wooden 3" x 8" nailing strips to each stringer. The top deck is then nailed to the nailing strips, thus providing continuous lateral support.

In order to determine the effectiveness of this type of bracing, a series of laboratory tests was conducted. It was the purpose of these tests to accurately evaluate the strength of the stringer members and thus provide an alternate rating.

2. Model Descriptions

Two models were constructed for test purposes. One model consisted of three 6I12.7, 10.0' long steel girders, spaced at 3.0 feet. The three girders were welded at each end to 3" x 3" steel angles. The other model consisted of the same members, but had nailing strips bolted to the side of each stringer. Attached to each strip were 3" x 10" wooden planks.

Each model was instrumented with SR-4 strain gages attached on the top and bottom flanges at midspan. Vertical deformation at midspan of the models was also measured during each test.

The models were subjected to two loads applied on the top flange of the center girder. The model which did not have nailing strips did have similar planking laid across the model width.

3. Results

The resulting stress data indicates that the average distribution is 14% - 72% - 14% for the model without nailers and 22% - 56% - 22% for the model with nailers, relative to exterior-center exterior girders. The load distribution factor, using this information, yields a factor of S/4.2 and S/5.4 for the deck without and with nailers, respectively.

RECOMMENDATIONS

Each of the six bridges had been previously rated, and have the following load limits;

Longs Mill Road Bridge	H0
Daysville Road Bridge	H3
Water Street Bridge	H1.2
Gapland Road Bridge	H5
Old Hagerstown Road Bridge	H1
Crum Road Bridge	H2

All of these bridges have been studied in detail and a new set of ratings computed. These new ratings are based on the observed field live load stresses, the computer dead load stresses and strength afforded by the nailing strips on the floor.

A summary of these results is given in Tables 1 through 6 for the six bridges. These results suggest the following ratings, providing the stringers and floor beams have adequate lateral supports, which means a new floor deck system:

Longs Mill Road Bridge	H10
Daysville Road Bridge	H5
Water Street Bridge	H10 (+)
Gapland Road Bridge	H8
Old Hagerstown Road Bridge	H10 (+)
Crum Road Bridge	H6

(+ = new floor system required)

The Daysville and Gapland Road Bridges, which have ratings less than H10, are governed by the truss elements. The Crum Road Bridge rating is governed by the floor beam. A comparison of these new ratings and the analytical ratings, as listed previously, shows a substantial increase in the load capacities of the six bridges.

TABLE 1

Bridge Rating
Longsmill Road Over Owens Creek

Member	Gage No.	Dead Load Stress (MN/m^2)	Live Load Stress (MN/m^2)	Allowable Stress (MN/m^2)	Rating
Floor Beam	1	20.7	82.8	[a]69.0	H8.8
Diagonal	2	0	32.4	86.3	H40
Diagonal	3	0	-33.8	86.3	H12.8
Stringer	4,5	-	68.3	86.3	H18.9
Botton Horizontal	6	33.1	18.6	86.3	H42.8
Botton Horizontal	7	33.1	22.8	86.3	H35

[a]Corrosion of member. 1 ksi = 6.9 MN/m^2 .

TABLE 2

Bridge Rating
Daysville Road Over Israel Creek

Member	Gage No.	Dead Load Stress (MN/m^2)	Live Load Stress (MN/m^2)	Allowable Stress (MN/m^2)	Rating
Stringer	1	6.9	72.5	86.3	H16.4
Botton Horizontal	2	27.6	20.7	86.3	H42.5
Top Horizontal	3	-19.3	-43.5	32.4	H4.5
Top Horizontal	4	-19.3	-35.2	32.4	H5.6
Floor Beam	5	24.2	147.0	82.8	H6
Vertical	6	0	12.4	86.3	H104

1 ksi = 6.9 MN/m^2 .

TABLE 3

Bridge Rating
Water Street Road Over Israel Creek

Member	Gage No.	Dead Load Stress (MN/m^2)	Live Load Stress (MN/m^2)	Allowable Stress (MN/m^2)	Rating
Diagonal	1	23.5	60.0	75.9	H13
Bottom Horizontal	2	37.3	33.1	86.3	H22
Stringer	3	8.3	51.8	86.3	H22.6
Diagonal	4	58.0	49.7	-	-
Floor Beam	5	14.5	18.6	[a]43.1	H23
Top Horizontal	6	-24.8	-31.1	58.7	H16

[a]Corrosion of member. 1 ksi - 6.9 MN/m^2 .

TABLE 4

Bridge Rating
Gapland Road Over Broad Run

Member	Gage No.	Dead Load Stress (MN/m^2)	Live Load Stress (MN/m^2)	Allowable Stress (MN/m^2)	Rating
Stringer	1	5.7	45.5	86.3	H26.7
Floor Beam	2	20.0	66.2	86.3	H11.5
U Bolt	3	-	70.4	-	-
[a]Vertical	4	12.4	33.1	43.1	H14
[a]Vertical	5	12.4	2.1	43.1	H225
Bottom Horizontal	6	31.1	24.8	86.3	H33
Diagonal	7	33.8	66.2	69.0	H8

[a]Corrosion of member. 1 ksi = 6.9 MN/m^2 .

TABLE 5

Bridge Rating
Old Hagerstown Road Over Little Catoctin Creek

Member	Gage No.	Dead Load Stress (MN/m^2)	Live Load Stress (MN/m^2)	Allowable Stress (MN/m^2)	Rating
Bottom Horizontal	1	35.9	10.4	86.3	H73
Diagonal	2	26.9	51.8	86.3	H17
Stringer	3	13.1	[b]134.6 (69.)[c]	86.3	H8 (15.9)[c]
Stringer	4	13.1	[b]117.3 (69.)[c]	86.3	H9.4 (15.9)[c]
Stringer	5	13.1	[b]183.5 (69.)[c]	86.3	H6.5 (15.9)[c]
Floor Beam	6	29.0	86.9	86.3	H9.9

[b]Without lateral support.
( )[c]With lateral support, new nailers.
1 ksi = MN/m^2

TABLE 6

Bridge Rating
Crum Road Over Israel Creek

Member	Gage No.	Dead Load Stress (MN/m^2)	Live Load Stress (MN/m^2)	Allowable Stress (MN/m^2)	Rating
Top Chord	1	-22.1	-39.3	86.3	H9.9
Floor Beam	2	42.8	107.6	86.3	H6
Stringer	3	6.2	58.0	86.3	H20.7
Vertical	4	10.4	26.9	86.3	H42
Bottom Horizontal	5	35.2	43.5	86.3	H17.6
Bottom Horizontal	6	31.0	39.3	86.3	H21
Top Chord	7	26.2	20.7	86.3	H44

1 ksi = 6.9 MN/m^2 .

BRIDGE EVALUATION BY FIELD TESTING

Baidar Bakht,
Ontario Ministry of Transportation and Communications,
Downsview, Ontario, Canada

Leslie G. Jaeger,
Technical University of Nova Scotia
Halifax, N.S., Canada

Tarun Naik
The university of Wisconsin-Milwaukee
Milwaukee, Wisconsin, U. S. A.

SUMMARY

Theoretical analysis of the load carrying capacity of highway bridges has been shown to underestimate significantly the actual strengths of bridges. This underestimation arises from several factors, not all of which can be eliminated by going to more refined methods of analysis.

The paper shows bridge testing to be an invaluable tool for establishing safe load carrying capacities of existing bridges. Through examples of tests on actual bridges, it is shown that load carrying capacities established through a test are usually significantly more than those obtained by theoretical evaluation.

The paper also discusses briefly the mechanics of the bridge testing operation.

INTRODUCTION

An extensive bridge testing program in Ontario, involving more than 150 bridges over the past 10 or so years (Bakht and Csagoly, 1979; - 1980; Bakht, 1981; Holowka and Csagoly, 1980; Taylor and Holowka, 1978; etc.) has conclusively proven that most bridges possess far more reserve strengths than they are given credit for on the basis of the usual evaluation techniques (e.g. AASHTO, 1977; CSA. 1974). The conclusion regarding very great reserve strengths in bridges can also be drawn from the fact that although most bridges in Canada and the U.S.A. are designed according to the same AASHTO specifications, yet these bridges are subjected to vehicles governed by a diversity of vehicle weight regulations. The diversity is such that the maximum vehicle weight permitted in one jurisdiction can be up to 100% more than that permitted in another jurisdiction. It is obvious that the diversity in permissible vehicle weights is made possible by great reserve strengths in bridges. Jurisdictions permitting heavier vehicles, such as Ontario, are clearly making use of some of the reserve of strength.

Bakht and Csagoly (1981) suggest that approximations or inaccuracies related to the following factors are mainly responsible for the inability of usual evaluation techniques to correctly determine the load carrying capacities of bridges:

a. force analysis;
b. strength analysis;
c. basic assumptions regarding the behaviour of certain components;
d. interaction of various components;
e. actual material strengths;
f. evaluation philosophy.

Approximations in two of these factors, namely force analysis and strength analysis, can be reduced, to a certain extent, by advanced analytical techniques; representative values of material strengths can be obtained by a thorough material testing program; and the shortcomings in the evaluation philosophy can be overcome by a rational and progressive code, such as the Ontario Highway Bridge Design Code (1979). However, in some cases, errors in the basic assumption regarding the behaviour of certain components, and uncertainties in the degree of interaction of the various components cannot be established except by a test on the bridge.

In this paper it is argued that in some cases errors pertaining to factors c and d can have a significant effect on the strength of a bridge. In such cases evaluation by a load test is proposed to be the only practical means to establish the optimum load carrying capacity of the bridge.

This paper, besides discussing the mechanics and equipment requirements of a test setup, also provides the outline of a procedure for obtaining representative load carrying capacities from proof loads.

BRIDGE TESTING

The process of load testing bridges, especially new ones, to ascertain whether the structure is "sound" is centuries old. This process usually consisted of subjecting the bridge to uniformly distributed static loads representing the "working loads". If the bridge did not collapse, or show excessive deflections, it was considered to be a sound structure. In recent times bridge testing has been used for a variety of other purposes. ASCE (1980) classifies bridge testing into the following five types:

a. Behaviour tests, (including dynamic tests) required to verify analytical results.

b. Ultimate load tests, required to confirm predicted ultimate loads (e.g. Sanders et al, 1975).

c. Stress history tests, required to establish the distribution of stress ranges in fatigue prone areas of bridges (e.g. Bakht and Csagoly, 1980).

d. Diagnostic tests, required to determine the cause of distress in a structure (e.g. Bakht and Csagoly, 1980).

e. Proof tests, required to establish the safe load carrying capacity of a bridge (e.g. Bakht, 1981).

The last mentioned type of test, i.e. a proof test, is required for bridge evaluation. In this type of test, the structure is subjected to static loads which are greater than, or equal to, the maximum allowable loads on the bridge multiplied by the appropriate load factors and dynamic load allowance (impact factor).

Equipment Required

Since the loads applied in a proof load test are exceptionally high, it is not considered safe for personnel to remain in the vicinity of the structure during the testing operation. Therefore, all the loading and response measuring equipment must be capable of being handled remotely from the bridge.

A structure can be loaded either by ballast, e.g. concrete blocks, placed on the structure by means of a crane (as shown in Fig. 1), or by trucks the axle weights of which can be regulated by the ballast that they carry.

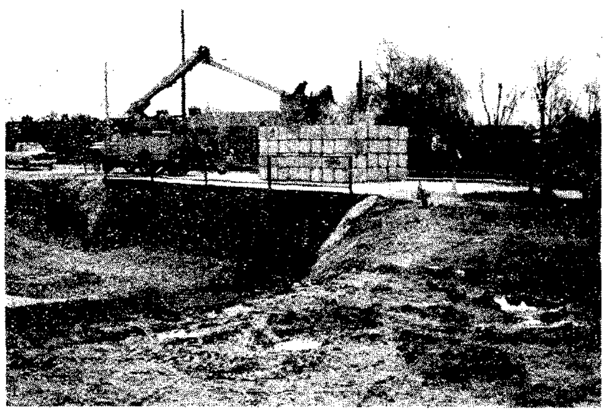

Fig. 1 Loading of a bridge with concrete blocks

Of the many disadvantages of the ballast-blocks type of loading the main one is that the load cannot be removed quickly from the bridge in case of a distress warning. The lack of manourability of the ballast blocks also inhibits testing of the structure for various locations.

This type of loading is especially inappropriate for bridges which undergo high strain changes with temperature variations. Because the loads are applied for a relatively long period of. time, the measured responses in such bridges are usually a result of the combinations of live load responses and temperature induced responses, and the combination is such that it is difficult to separate the two effects.

A loaded vehicle, representing heavy commercial vehicles, is an ideal means of applying loads to a bridge during the proof testing operation. Figure 2 shows two such vehicles on a bridge. The weights of these vehicles are controlled with concrete blocks. Each concrete block weighs about one tonne. The vehicles, which are equipped with self loading cranes, have five axles and are capable of carrying gross weights of about 95 tonnes. The vehicles are also capable of being driven by remote control.

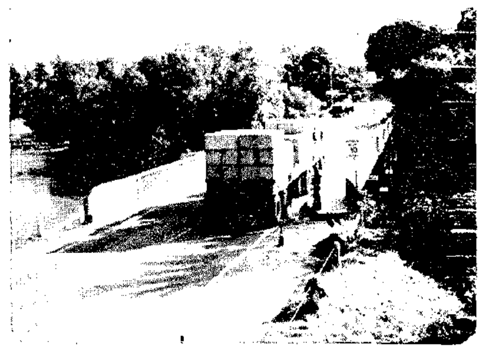

Fig. 2 Loading of a bridge with vehicle loaded with concrete blocks

There are usually two kinds of response that are measured during a proof load test: viz. deflections and strains. Because of the necessity of staying well away from the bridge during the testing operation, these responses must be measured remotely. Deflections can be conveniently measured by means of deflection transducers or high precision levels. Strains in steel members can be measured by means of standard electrical resistance strain gauges. However, measurement of strains in concrete and wood members require specially manufactured strain transducers, such as the one shown in Figure 3a. This transducer magnifies the actual strains for easy recording. Figure 3b shows a dismountable strain transducer which is suitable for strain measurement in steel components. Both of these transducers were developed by the Ontario Ministry of Transportation and Communications for bridge testing operations (Wagner, 81; - ,81)

The minimum requirement for data acquisition is a portable strain meter using which strains and deflections are recorded manually. When large numbers of strains and deflections are to be recorded, manual reading with the portable strain meter can dangerously slow down the proof load testing operation, in which case it is advisable to use an automatic data acquisition system, such as the one shown in Figure 4a. The system used for the Ontario Ministry of Transportation and Communications bridge tests is housed in the mobile laboratory shown in Figure 4b.

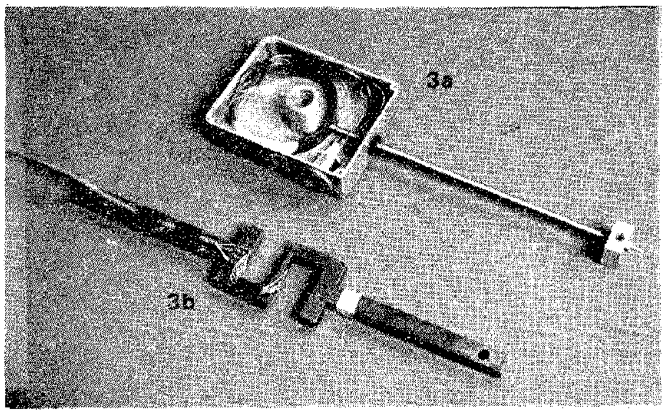

Fig. 3a A demountable strain transducer for concrete and wood

Fig. 3b A demountable strain transducer for steel

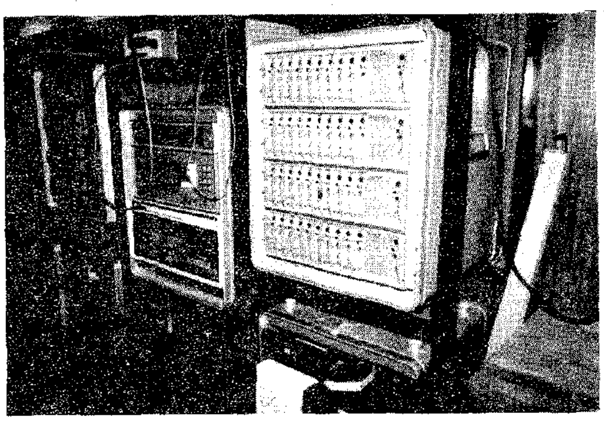

Fig. 4a A computer-based data acquisition system for static load tests

Fig. 4b A mobile laboratory

BASIC ASSUMPTIONS

No degree of refinement in calculations can compensate for shortcomings in the basic assumptions regarding the behaviour of certain components. The basic assumptions of some theories that are still in use in bridge design, are open to question. These theories have resulted in a vast reserve of strength, so far untapped, in bridges.

One particular example is that of deck slabs in slab-on-girder bridges. Extensive testing of models of concrete deck slabs and deck slabs of existing bridges has conclusively established the presence of an arching action, which is usually ignored in design because of the difficulties of incorporating it even in otherwise sophisticated computer programs (Csagoly, 1979; Csagoly et al, 1978). It is noted that due to this arching action, the load carrying capacity of deck slabs supported by girders and subjected to concentrated loads is considerably more than that governed by flexure alone. The Ontario Highway Bridge Design Code (1979) provides charts by the use of which realistic estimates of the load carrying capacities of various deck slabs can be obtained.

The arching action is also present in concrete beams. However, the degree of arching action cannot be established by generally available analytical tools. The surest way of taking advantage of this arching action in the evaluation process is by testing the beam. The test on a beam, or the deck slab, consists of application of a concentrated load, the intensity of which is gradually increased and decreased, and the measurement of the deflection of the point of load application.

Fig. 5 An open spandrel arch bridge

To give an example of the procedure, some details of the test on a concrete arch bridge shown in Figures 5 and 6 are presented. The floor beams of the bridge, which consist of steel I sections encased in concrete, were considered to be the weakest components in the bridge. Because of these floor beams the bridge was posted for nearly 7 tonnes. To obtain a realistic estimate of the strengths of the floor beams it was decided to test them with the punching shear device attached to the underside of one of the testing vehicles shown in Figure 2.

The punching shear device, described by Csagoly (79), applies loads to two 254 mm x 254 mm pads 152 mm apart through an hydraulically operated plunger located on the underside of the vehicle.

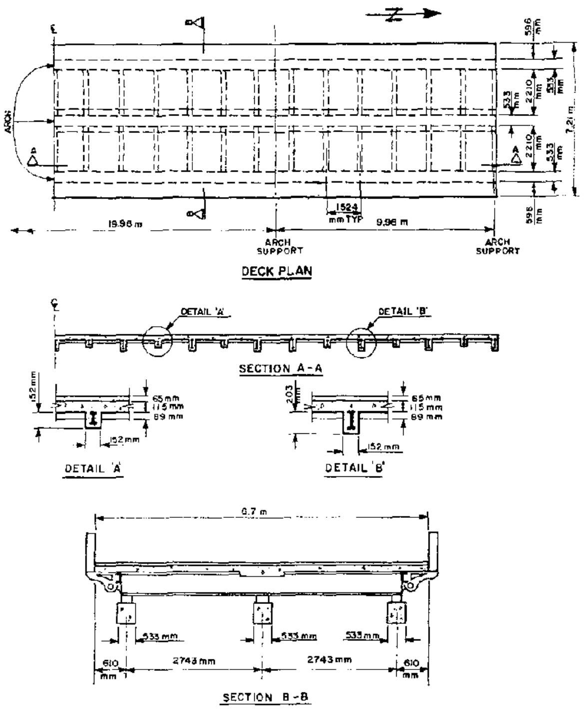

Fig. 6 Flooring system of the arch bridge

The plunger is centrally located at a distance of about 2.9 m from the nearest wheel of the vehicle. The applied load is measured through a deflection transducer. The test consists of the gradual application and releasing of the load on the two pads, and monitoring the load - deflection curve which is simultaneously drawn on an x - y plotter mounted on the vehicle. Loads up to which the response of the slab remains elastic, are regarded as safe loads for the slab. A photograph of the punching shear device is shown in Figure 7.

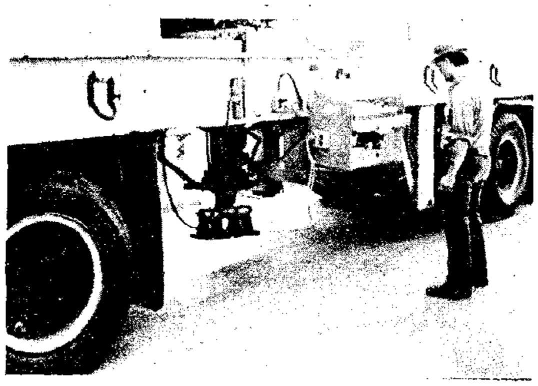

Fig. 7 A punching shear device mounted under the test vehicle

Two of the observed load - deflection curves for tests on floor beams are shown in Figure 8 together with a curve for test on a slab panel. It can be seen that the floor beams behaved substantially similarly to the deck slab, with the only difference being that the beam deflections were somewhat smaller than the slab deflections.

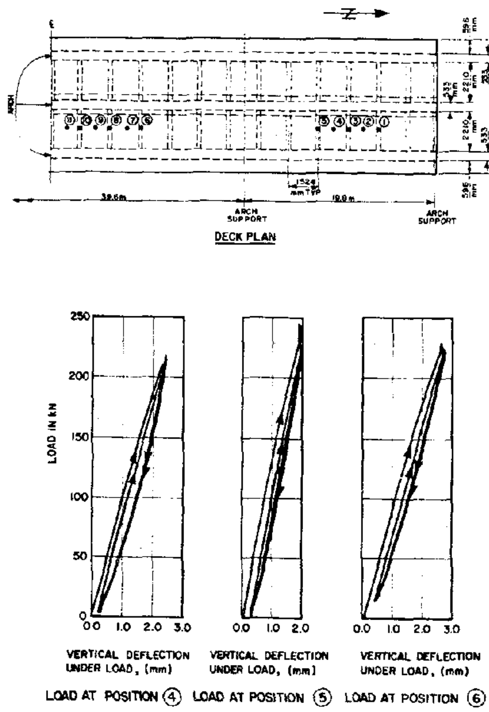

Fig. 8 Load-deflection curves under a concentrated load

From the punching shear tests it was concluded that the floor beams and the deck slab were capable of sustaining static wheel loads of 54 tonnes, or a moving axle weight (assuming 50% dynamic magnification) of nearly 70 tonnes. It is noted that the maximum axle weight permitted anywhere in Canada and U.S.A. is only 24 tonnes.

Flaws in basic assumptions can be readily eliminated by a carefully planned and executed test.

COMPONENT INTERACTION

Neglect of interaction between the various components of a bridge can sometimes lead to significant underestimation of the load carrying capacity of the structure. Rigorous analysis can be used to establish the effect of interaction in a qualitative manner, but a quantitative assessment of the degree of interaction requires exact knowledge of actual boundary conditions. Since in many cases it is almost impossible to assess the true boundary conditions by mere inspection, the rigorous analysis would prove to be of only limited value. In such cases measurements of strains or deformation at strategic locations, under known live loads, can prove very useful in the evaluation process.

One example of uncertainty in the degree of component interaction is that of the composite action between a steel girder and the concrete deck slab, when there are no mechanical shear connectors. During the various bridge tests undertaken in Ontario (Bakht and Csagoly, 1979) it was observed that even in the absence of intentionally provided shear connectors, there existed full composite action between the slab and the steel girder, provided that the top flange of the girder was partially embedded in concrete, and that the interface between the concrete and steel was sound. Clearly, there could be uncertainty regarding the soundness of the interface. In such cases a few strain gauges installed at the top and bottom flanges of a girder, can remove doubts regarding the degree of composite action.

Fig. 9a Columns in a bridge

As another example of uncertainty in the degree of component interaction, a long column of a twin girder bridge, shown in Figure 9a, is presented. The connection detail at the base of the column was as shown in Figure 9b.

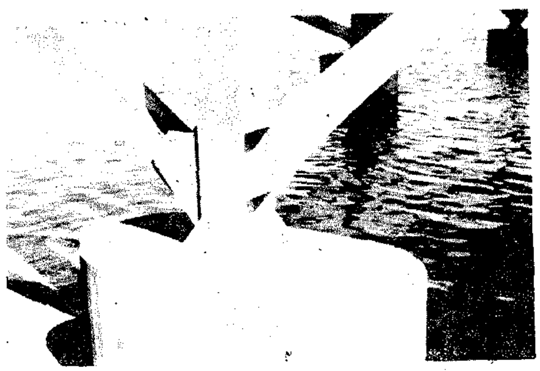

Fig. 9b Details of base connection of a column

When the bridge under consideration was evaluated the column, following the usual practice, was assumed to be hinged at the bottom. In addition, it was assumed that the loads from the girder were transferred to the column with an eccentricity equal to the distance between the vertical line of bolts and column centre line. The live load moments caused by the assumed load eccentricity, coupled with the assumption of column being pinned at the lower end, suggested that the columns were unsafe to carry normal live loads.

A few columns of the bridge were fitted with electrical resistance strain gauges at various locations, and the response of the gauges monitored under the test vehicles. The typical response of a column under live load, as shown in Figure 10, suggested that the columns were almost completely fixed at their bottom ends, and that the live load eccentricity was always less than 65 mm, compared to the assumed eccentricity of nearly 200 mm. The base plates, which were connected through welded gusset plates to the column flanges, had foundation bolts situated on either side of the flanges, thus greatly restricting the rotation of the columns feet in the plane of buckling.

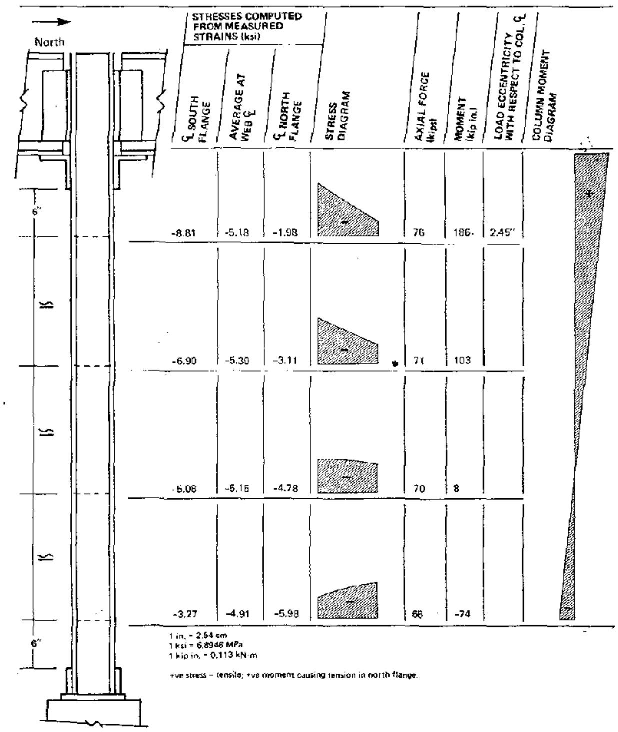

Fig. 10 Observed live load response of a column

PITFALLS IN TESTING

The proof load testing of a bridge is both an art and a science. It requires a thorough understanding of the structural behaviour of the bridge, and a pure "gut feeling" regarding its capacity to sustain loads and the nature of its weakest links. The proof load test is reached after the bridge has been subjected to lesser loads and the response of structure has been rationalized. Sometimes it is necessary to perform a behaviour test before planning the proof load test. The data from the behaviour test are analysed, mainly to locate those areas of the bridge which must be monitored closely during the proof test. Sometimes, when such data are not available, reliance is placed on linearity of data from previous load lifts, and on the elasticity of responses. In any event the proof load testing operation is a risky undertaking, since there is always a possibility of the structure failing under the test loads. It should, however, be noted that under careful supervision the possibility of failure is quite remote. More than 150 bridges have been tested in Ontario without a mishap.

The fact that a bridge has been subjected to a high load without showing any sign of distress does not mean that the bridge can carry such loads at all times. The test on the bridge shown in Figure 5, is presented to illustrate the point. The response of the bridge without any live load was monitored over a 24 hour period at regular intervals to study the reaction of the structure to changes in temperature. It was found that due to a temperature drop the arches were subjected to downward forces resulting from the interaction of the arches and the flooring system.

It is obvious that the bridge may or may not be able to sustain the same vehicle weights in winter as it can during the summer. It is noted that the proof load test on the bridge was done in the summer when temperatures ranged between 15^{o} and 30^{o}C. From the temperature study it was concluded that normal traffic could be permitted on the bridge for temperatures down to -10^{o}C. For lower temperatures the bridge was recommended to be closed to heavy commercial vehicles.

DYNAMIC LOAD ALLOWANCE

The actual value of the dynamic load allowance (DLA) or impact factor, has a considerable influence on the intensity of loads that could be permitted on a bridge. The value of DLA, however, depends not only on the dynamic system of the bridge, but also on the dynamic system of the vehicle. If the natural frequencies of the bridge and the vehicle are close a high value of DLA results: for other cases the value of DLA is smaller (OHBDC, 1979). Hence for a given bridge the representative value of the DLA can be obtained only by monitoring the dynamic response of the structure over a representative period of time under normal traffic.

It is noted that for almost continuous recording of strains and deflections the equipment must be a computer based system. Details of the equipment requirement are given by Bakht and Csagoly (1979).
Although the DLA obtained by using the test vehicle may not be representative, in some cases the test with even a single vehicle can yield important information regarding the dynamic characteristics of the bridge. For example a dynamic test on a spandrel filled reinforced concrete arch bridge showed that when the riding surface of the bridge was smooth, no dynamic magnification of load effect took place. However, when the test vehicle passed over an artificially created irregularity on the riding surface, some magnification of load effects resulted. This phenomenon is also observed in soil-steel structures (Bakht, 1981). Figure 11 shows some of the observed time-strain curves in a soil-steel structure. Because of the observations mentioned above it was possible to use a much smaller value of the DLA for reinforced concrete arch bridge than is usually required for design.

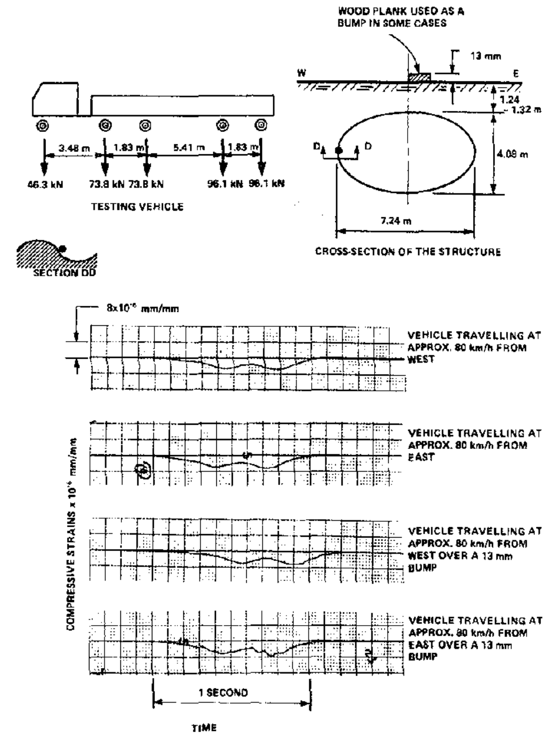

Fig. 11 Observed dynamic response of a soil-steel structure

DETERMINATION OF TEST VEHICLE LOADS

In this section of the paper a method for determining the loads of the test vehicle is presented.

It is presumed that the bridge is being evaluated on the basis of a known design (or evaluation) vehicle loading. Typically this design loading may consist of a number of concentrated loads, a combination of concentrated and uniformly distributed loads, or a uniformly distributed load only. It is supposed further that there is available a test vehicle, so that the spacings of the wheel loads are already known and the size of some or all of these loads is to be determined. In establishing the concentrated loads for the test vehicle it is recommended that the following criteria be met.

(a) overall behaviour of the bridge for bending moment and shear force.

The overall behaviour of the bridge in relation to bending moment and shear force is used to establish an equivalence between the static loading of the testing vehicle and the loading of the design vehicle as enhanced by the load factor and dynamic load allowance.

Recommendation: That the wheel loads of the testing vehicle be so chosen that at every section of the bridge the maximum bending moment and maximum shear force due to passage of the test vehicle shall be equal to or greater than those due to the design vehicle as enhanced by the appropriate load factor and dynamic load allowance.

It is noted that in the Ontario Highway Bridge Design Code (1979 the load factor is 1.4; the dynamic load allowance is typically 0.35, so that the multiplication factor for this effect is 1.35. Hence the overall factor by which the design vehicle loads must be multiplied is typically 1.4 x 1.35 = 1.89.

(b) <u>local punching shear behaviour</u>

This frequently governs the strength of a deck slab and some floor beams. The punching shear aspect may or may not have been covered in the <u>design</u> vehicle by provision of appropriate concentrated loads. However, even if the design vehicle is defective in this regard (for example - if the design load is simply a uniformly distributed load) it is absolutely necessary for the test vehicle loadings to take it into account.

<u>Recommendation</u>: That the load applied to the bridge by the punching shear device of the test vehicle (e.g. that shown in Figure 7) be equal to or greater than the maximum single wheel load permitted by the jurisdiction, as enhanced by the appropriate load factor and dynamic load allowance.

It is noted that the dynamic load allowance for a single wheel is not necessarily the same as that for an entire vehicle, and indeed is usually somewhat higher. In the absence of specific code instructions it is recommended that the dynamic load allowance be taken as 0.5. Hence the overall factor by which the single wheel load would be multiplied according to the Ontario Highway Bridge Design Code is 1.4 x 1.5 = 2.1.

(c) <u>width of the test vehicle</u>

In establishing equivalence between the test vehicle and the design vehicle in (a) above it has been implied, but not stated, that the two vehicles are of the same width. If this is not the case then allowance must be made for the difference. Clearly a narrow test vehicle, having two lines of wheel loads closely spaced, produces a more severe loading condition on the bridge than the same two lines of wheel loads spaced farther apart. The effect of width is important and should not be ignored; for example a bridge may have been designed originally for such special purpose vehicles as army tanks, which may be of markedly different width. It is possible to allow for width variation as between the test vehicle and design vehicle by simple approximations, but space does not permit an elaboration of these in the present paper.

CONCLUSIONS

Bridge testing is invaluable, when skillfully performed both as an addition to theoretical analysis and also (as in the bridge shown in Figure 12) in cases where accurate analysis is virtually impossible.

The proper design of the bridge test or series of bridge tests requires a clear understanding of what information is being sought and the way in which that information may be derived from the results. The interpretation of the results requires an appreciation of such factors as temperature conditions, and their effect on bridge behaviour.

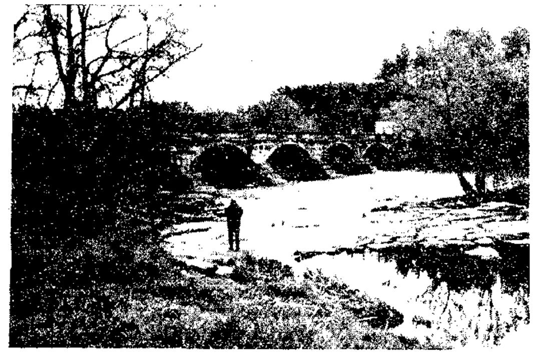

Fig. 12 A stone arch bridge

Fig. 13 A posted bridge during a test

A major advantage of a successful proof test is that it is frequently possible to increase the permitted loads which may cross the bridge to well above the levels obtained by usual evaluation techniques. This conclusion is illustrated in Figure 13 which shows a bridge posted for 10 tonnes and carrying two test vehicles each weighing about 80 tonnes.

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BRIDGE RATING AND COMPUTERIZATION

C. P. Heins.

Professor, Institute for Physical Science and Technology
Civil Engineering Department
University of Maryland
College Park, MD 20742 USA

SUMMARY

The rating of bridges requires a structural analysis of the system, when the bridge is subjected to assumed dead and live loads. These analyses may be complex or simple, depending on the structural type. In order therefore, to expedite such analyses, a series of computer programs has been developed and is available for direct use by the engineer.

It is therefore the purpose of this paper to present details of these computer programs.

INTRODUCTION

1. General

The rating of bridge structures first requires collection of the bridge characteristics as obtained from a field survey. After these data are collected, the bridge must be analyzed using appropriate analytical schemes. The engineer is thus faced with voluminous data and possibly complex structures to analyze.

This task can therefore be greatly minimized if proper computer software is available. It is therefore the intention of this paper to present details of presently available programs now being used by the engineering community.

2. Computer Programs

At present there are four general purpose computer programs that are available to the public for rating of bridges. Each program has certain features, as will be described later, that can accomodate specific bridge types. These programs are designated as follows:

i) Bridge Rating and Analysis Structural System (BRASS),
ii) Computer System for Analyzing and Rating Bridges (BARS),
iii) Bridge Analysis and Rating (BRRAT),
iv) Overload Route System (OVLOAD).

PROGRAM DETAILS

The details associated with each of these four computer programs, relative to capability, method, limitations, programming language, input/output, hardware and availability, will now be presented.

1. Bridge Rating and Analysis Structural System (BRASS)

Categories: bridge rating, highway bridge design, highway bridge review, bridge engineering computer programs.

Authors: Ralph R. Johnston, Robert H. Day and Dan A. Glandt; Wyoming Highway Department, P.O. 1708, Cheyenne, Wyoming, 82001.

Maintenance: Authors.

Date: September 1973.

Capability: System includes bridge design, structural inventory, deck design and review, structural analysis, structural loading, and girder section design and review. The types of structure are divided into six groups. The groups include: cell type layout, continuous beam bridge, rigid frame, slant leg structure, and rigid frame box culverts. The system is capable of analyzing reinforced concrete, structural steel and timber.

Method: The method of analysis for beam is column analogy. The method used in the analysis for cell structure or slant leg rigid frame is slope-deflection method. Gauss-Jordan inversion is used to invert the matrix vitrual work method as applied to calculate deflection.

Limit: The system only considers working stress method. It cannot analyze hinge and cantilever structure and at least one truck wheel has to be applied at the structure.

Programming Language: FORTRAN.

Documentation: Complete and current.

Input: The system consists of design review and load rating subsystem. The executive control has the duty of determining the next job. The input cards, called control cards, are followed by data cards which determine the phases to follow in a given component.

Output: The output consists of verification of input data, design reports and rating reports. The first report is always printed, but the last two are printed only if requested. The user may control output desired. That is, by coding, he can obtain intermediate output, along with final results.

Software Operating: Batch.

Hardware: The programs are coded on the IBM 360/40. Utilizing a storage capacity of 72K bytes. May be run on most 3rd generation computers.

Usage: The system is developed by the Wyoming Highway Department and used by several state agencies.

Availability: Through Wyoming Highway Department or U.S. Department of Transportation - Federal Highway Administration.

2. Computer System for Analyzing and Rating Bridges (BARS)

Categories: bridge analysis, rating, girders, truss, live load.

Authors: Control Data Corporation, P.O. Box 1980, Twin Cities Airport Station, MN 55111.

Maintenance: Authors.

Date: Spring 1972.

Capability: The system can perform the inventory ratint, operating rating, posting rating and special permit analysis. It can analyze five member types which are decking, stringer, floor beam, girder and trusses. Construction materials may be one of the following: structural steel, reinforced concrete, composite steel, prestress concrete, composite prestress concrete.

Limitations: Only allowable stress method is considered here. Live loads have limited to five truck types.

Programming Language: FORTRAN.

Documentation: Complete and current.

Input: The input consists of the geometries, properties of members, materials, and load data. System is production oriented and designed to batch process bridges of the same type. The first sheet summarizes the results while additional sheets provide detailed information.

Output: The first sheet for a bridge provides summary information indicating the rating of the bridge. Following the summary sheet are data sheets for each member rated providing the detail that supports the rating for that member. In addition to that, a tape file is written that will retain the data for the bridge that will be needed for insurance of special permits or for future ratings due to increased legal load limits.

Software Operating: Batch or RJE to CDC 6600. Batch or O.S. to IBM 360/50.

Hardware: The system is programmed for an IBM 360/50 having 512K primary storage.

Usage: The system is programmed for Illinois Division of Highways and used by several other state agencies.

Availability: Via service station of Control Data Corporation.

3. Bridge Analysis and Rating (BRRAT)

Categories: bridge rating, live load, rating factor, girder, floor beam, stringer, truss.

Authors: Pennsylvania Department of Transportation.

Maintenance: Authors.

Capability: Program calculate inventory, operating and safe load capacity ratings for single span concrete slab or T-beam bridges, multigirder composite or noncomposite bridges, girder-floor beam-stringer system or truss-floor-beam-stringer system. Ratings are computed for H20, HS20, ML70, Type 3S2 and Type 3-3 loadings and may be requested for maximum of 24 axles.

Method: The method of analysis is matrix method and Gauss-Jordan inversion.

Limit: The ratings are limited to working stress method. No hinge or cantilevel structures are able to analyze.

Programming Language: FORTRAN.

Documentation: Complete and current.

Input: The data to be entered depends on the type of btidge to be analyzed. Input consists of multigirder system (GGG), girder-floor beam-stringer system (GFS), truss-floorbeam-stringer system (TFS) and concrete slab or T-beam bridge.

Output: The output also printed out according to above four categories to have inventory, operating and safe load capacity rating factors in a fixed form.

Software Operating: Batch or remote.

Usage: The system is developed by Pennsyvlania Department of Transportation.

Availability: Via Pennsylvania Department of Transportation.

4. Overload Route System (OVLOAD)

Categories: bridge rating, highway route, overload, live load.

Authors: Kenneth R. White, John Minor, Department of Civil Engineering, New Mexico State University, Box 3CE, Las Cruces, NM 88003.

Maintenance: Authors and New Mexico State Highway Department.

Date: 12 September 1974.

Capability: The program OVLOAD can be used to check a proposed routing of a given overload (truck of object) requiring an overload permit) within the State of New Mexico utilizing the data bank created by the bridge inspectors of the New Mexico State Highway Department.

Method: Matrix stiffness method is used where one span is one member.

Limitation: Working stress method is applied here. No variation of member, hinge or cantilevel structures are considered. No moment or shear influence lines are constructed where only critical point is given.

Programming Language: FORTRAN.

Input: OVLOAD is operational on the CMS terminals connected to the NMSHD IBM 370. The bridge data which was accumulated with the NMSHD bridge inventory system is stored on disk within the CMS system. The data bank is periodically updated by the ECU from the master file: The program OVLOAD is accessed and used by the executive program OVERLOAD which sets up the necessary files for execution.

Output: Page one is a listing of all input data. Succeeding pages list the output data which are lists of all inadequate bridges.

Software Operating: CMS terminal to IBM 360/65.

Hardware: IBM 360/65, tape reader, teleprocessing unit having 126K core.

Usage: OVLOAD was developed as a joint project between the New Mexico State Highway Department and the New Mexico State University.

Availability: New Mexico State Highway Department.

COMPARISONS

In order to examine the relative versatility of each program, three tables have been developed.

As shown in Table 1, each program name, source, description, date available and computer type and language are given. As can be seen, each program is available for direct use on the IBM 360, and is written in FORTRAN IV.

The selection of the appropriate computer program can be determined by examining Table 2. This table gives the bridge type that can be analyzed, the construction material and the general program capabilities.

Associated with each bridge type are specific restrictions, i.e. number of spans, etc. Such restrictions or capabilities are illustrated in Table 3. Also given in the table is the analytical procedure used in analyzing the given bridge type.

In rating each bridge type, certain information is desirable. As shown in Table 4, for the four program types, characteristics such as live load functions, brudge properties and stress are catagorized.

CONCLUSION

The need for automating the load rating of various bridge types is paramount when a vast number of bridges are to be evaluated and their configurations vary.

In order to expedite such ratings, a series of computer programs have been developed for direct use by the engineer.

This paper presents the details of such computer programs and their availability.

TABLE 1. PROGRAM DATA

NAME	SOURCE	DESCRIPTION	DATE AVAIL.	COMPUTER	LANGUAGE
Brass	Wyoming State Highway Department	User may design, review or load rate structure. Load rating portion of the section analysis component load rates on shears, flexural stresses and bearing stresses using working stress method.	9/73	IBM 360/40	FORTRAN IV
BARS	Control Data Corporation	The system can perform the inventory and operating rating required by AASHTO specifications and the posting rating and special permit analysis required by state law using working stress method.	3/72	IBM 360/50 CDC 6600 (IBM 360/50 with secondary storage position) (IBM 1130 remote)	FORTRAN IV
BRRAT	Pennsylvania Department of Transportation	This program calculates Inventory, Operating and Safe Load capacity. Ratings on flexural stresses using working stress method.	9/74		FORTRAN IV
OVLOAD	New Mexico State	This program determines a capacity rating for all bridges on a route, determined by route number and DOD Section Number, within or across New Mexico.	9/74	IBM 360/65	FORTRAN IV

TABLE 2. PROGRAM DATA

NAME	BRIDGE TYPE								CONSTRUCTION MATERIAL						GENERAL CAPABILITY			
	Concrete Slab	Concrete T-beam	Steel Girder	Stringer Floor Beam	Truss	Box Culverts	Rigid Frame	Slant Leg	Structural Steel	Reinforced Concrete	Composite Steel	Prestressed Concrete	Composite Prestressed	Timber	Analysis	Design	Rating	Routing
BRASS	X*	X*	X	O	O	X	X	X	X	X	X	O	O	X	X	X	X	O
BARS	X	X	X	X	X*	O	O	O	X	X	X	X	X	O	X	O	X	O
BRRAT	X*	X*	X	X	X*	O	O	O	X	X	X	O	O	O	X	O	X	O
OVLOAD	X	O	X	O	X	O	O	O	N	N	N	N	N	N	O	O	X	X

*Limited to single span.

N = Not knowing what kind of material, mass storage only provided for section properties.

TABLE 3. ANALYSIS CAPABILITY

NAME	SIZE LIMITS			METHOD	LOAD TYPE					GENERAL CAPABILITY							
	Number Spans	Number Joints	Number Member		Max No. Live Load	Standard AASHTO LL	Special and General Truck Configuration	Load Train	Sidewalk LL	Cantilevers	Hinges	Nonprismatic Member	Flange Transition	Composite or Noncomposite	Load Factor Method	Working Stress Method	Tapered Section
BRASS	19	20/span	18/span	Column analogy Slope deflection	24	X	X	X	X	O	O	G	O	X	O	X	O
BARS	6	10/span	--	Matrix method	6	X	O	O	O	O	X	G	O	X	O	X	O
BRRAT	8	80	80		8	X	X	X	X	O	O	R	O	X	O	X	O
OVLOAD	50	50	50	Matrix stiffness method	50	X	X	X	O	O	O	R	O	O	O	X	O

G = General; R = Restricted

TABLE 4. AUTOMATIC CAPABILITY

NAME	LIVE LOAD								BRIDGE PROPERTIES					STRESSES			
	Moments	Shears and Reaction	Deflection	Impact Factor	Distribution Factor	Sidewalk LL	Moment Critical Point	Shear Critical Point	Composite	Noncomposite	Standard Section Table	Plate Girders	Section Properties	Yield Stress for Steel	Inventory Rating	Operating Rating	Load Factor Method
BRASS	X	X	X	S	I	I	X	X	X	X	O	X	IS	I	I	I	O
BARS	X	O	O	IS	S	O	X	O	X	X	X	X	IS	I	S	S	O
BRRAT	X	O	O	IS	I	I	X	O	X	X	O	X	I	I	S	S	O
OVLOAD	X	X	O	O	O	O	X	O	O*	O*	O*	O*	S	O*	S**	S**	O

I = Input, S = Self generated, IS = Input or self generated.

* = Section properties were stored in mass storage beforehand.

** Overload moment compared with standard HS load moment.

REHABILITATION OF PELHAM PARKWAY BRIDGE
IN DEPTH INSPECTION

Avanti C. Shroff

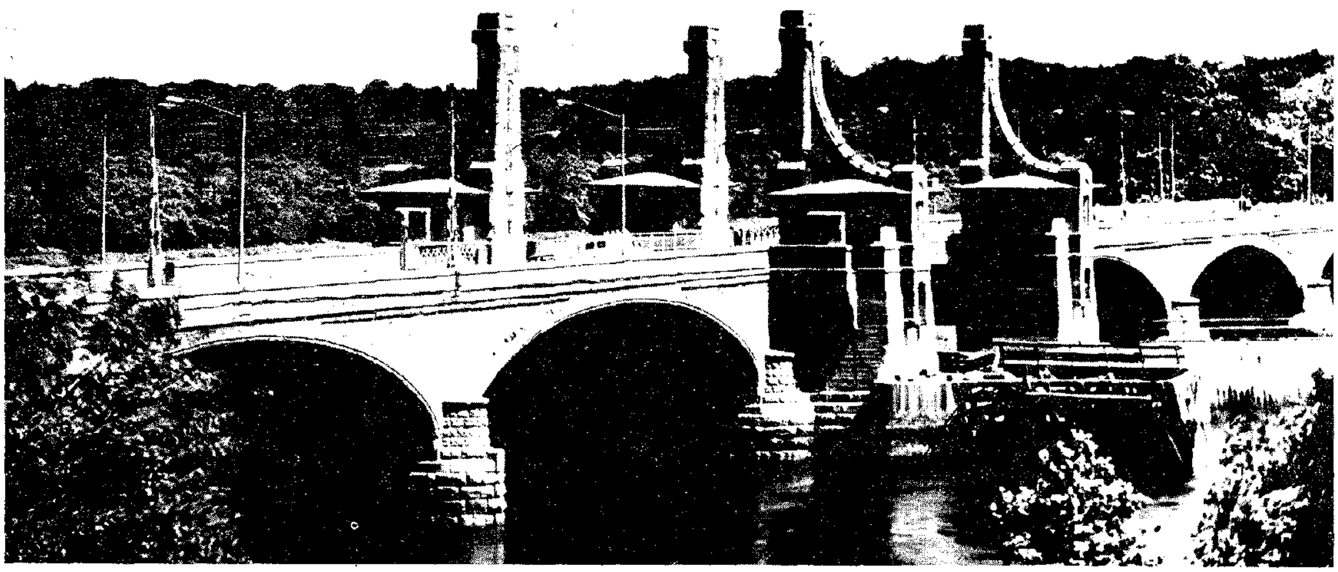
Vice President, Iffland Kavanagh Waterbury, P. C.

SUMMARY

The Pelham Parkway Bridge over Eastchester Bay in The Bronx, New York is a seven span, four lane structure constructed in 1907. The seven spans consist of a moveable span in the center and three fixed approach spans on each end. The moveable span is a double-leaf Scherzer Rolling Lift, bascule structure. The fixed spans are concrete spandrel arches with the roadway placed on fill. Overall length of the structure between ends of wingwalls is about 855 feet. The width of the bridge is 52 feet out to out of parapets.

The entire project was divided into four major phases of work namely: In-depth inspection of the entire structure, Stress analysis and rating of both the basic structure and bascule machinery, Preparation of reconstruction project report to include recommendations and construction cost estimates along with results of inspection and stress analysis and finally, Preparation of contract documents for the rehabilitation of the bridge.

This paper, the first one in a series, discusses in-depth inspection (Phase 1) of the basic structure exclusive of basule machinery and preparation of Reconstruction Report (Phase 3) of the project. The paper includes discussions on inspection equipment, methods and techniques utilized along with results of the inspection. The paper also describes an innovative method of utilizing photographic techniques in preparing an engineering inspection report.



GENERAL DESCRIPTION

Arch Spans

Each approach to the bascule span is made up of three concrete arch spans. The clear span of the arches is 105 feet (32m) at the spring line with a rise in the arch of 16.5 feet (5m). Arch sections vary from 2 feet (610mm) at the crown to about 5 feet (1.52m, normal) near the spring line. Embedded in each concrete arch are eighteen steel, arch truss ribs, 3 feet (910 mm) on centers. The trusses vary from 1.5 feet (460mm) deep at the crown to 5.17 feet (1.58m) at the ends, with chords composed of 2 angles - 3"X 3"X 3/8" and the diagonal web member of 1/4' X 1-3/4" flats with 2-angles - 3"X3"X3/8" as an end vertical. Trusses are connected at the crown and quarter points with 3"X3"X3/8" angle cross bracing. There is no other longitudinal reinforcement in the arch. The spandrels are unreinforced concrete gravity walls with a maximum height to the sidewalk level of about 10 feet (3m). There is a concrete end wall across each abutment at the ends of the wingwalls.

Fill over the arches varies from about 1 feet (300mm) at the crown to about 6.5 feet (2m) at the spring line (including the pavement).

Bascule Span

Each leaf of the bascule is composed of four trusses, 12.25 feet (3.7m) on centers. The trusses vary in-depth from 2 feet (610mm) at the crown to about 10.75 feet (3.3m) at the support. The trusses are 40 feet (12.2m) long from the center line of trunion to the centerline of the crown point with an 11.33 feet (3.5m) back span containing the counterweight and a 14 feet long fixed steel span beyond the end joint. Floor beams at 10.5 feet (3.2m) centers are framed to the trusses and support longitudinal stringers that in turn support the metal grating floor system. Vertical diaphragm frames brace the trusses at two panel points near the towers. Portions of the deck outside the exterior trusses are supported on cantilever floor beams brackets.

The piers of the bascule span rest on a continuous pile supported concrete mat foundation.

IN-DEPTH INSPECTION (PHASE 1)

Phase 1 of the project included in-depth inspection of the basic structure, including pavement, curb, sidewalk, drainage, spandrel walls, parapets, concrete arches, bascule span, towers, piers, abutments, fender system, waterway and marine traffic, approaches, sign structures and utilities, etc., as well as the Bascule Machinery including mechanical and electrical drive components, stabilizing and control components, etc.

Inspection Procedures

The inspection was performed by a team of 5 people comprising of a team leader (senior licensed professional engineer), an assistant team leader, a Senior Field Aid and two Junior Field Aids. The team was equipped with two vehicles, mini boards, flashing lights, traffic signs, traffic cones, fluorescent flags, etc., for maintenance of traffic during inspection. Vehicle drivers (Junior Field Aids) served as flagmen during maintenance of traffic operations. The vehicles also served for transportation of the inspection team and their equipment team to and from the bridge site. The team was furnished with survey equipment, pneumatic chipping guns, increment borer, cold chieseis, extension ladder, sonic thickness meter, chipping hammers, etc.

The original proposal was to use a rolling scaffold for close-up inspection of underside of the arches, bascule span, above water areas of piers and timber fender systems. The scaffold spanning across the width of the structure was to be hung from the parapet walls. However, for reasons of safety, economy and ease of inspection, a 50 foot work boat equipped with a platform supported on a pipe scaffold was used.

Underwater inspection of all pier surfaces (about 15,000 sq. feet-1,500 sq. meters at mean water level) and timber fender systems (about 2,400 sq feet-240 sq. meters) was performed by experienced professional divers specializing in underwater inspection of bridges or similar structures.

Coring Program

Supplementing the visual and "hands on" inspection procedures utilized throughout the structure, a drilling and coring program with appropriate follow-up testing was undertaken. Vertical drilling was done in the approach pavements at five locations and vertical drilling and coaring was undertaken in the bridge pavement and structure at thirty-six locations. The test program included Petrographic examinations, Compressive Strength tests, Air Content tests, Freeze-Thaw tests and Chloride tests.

RECONSTRUCTION PROJECT REPORT (PHASE 3)

One of the important phases of the project was to prepare an Reconstruction Project Report, containing results of in-depth inspection, stress analysis, photographs representative of each type of problem encountered, recommendations and construction cost estimates of rehabilitation work proposed.

This report was reviewed, in detail, by the client and the definition and scope of services for the Final Design Document Phase (preparation of estimates) were established.

Rehabilitation projects dictate significant amount of time to be spent in the field and office to properly evaluate the present condition of all structural elements. This, in turn, allows for required alternate design solutions for rehabilitation. Hundreds of photographs, sketches, details, reports, etc., are generated in the process. To compile all this mass of data into a single concise report such that a reviewer, without visiting the project, can clearly understand the entire process of evaluation, is an extremely difficult task. Hence, efforts were put in since the beginning of the project to develop some innovative methods to address this problem.

One of the methods considered for the purpose was photography. A photographs has some unique qualities. It can be very selective and can emphasize selected areas in great detail quickly and accurately. In a day or two, almost every square inch of a structure can be permanently recorded. With tools such as telephoto lenses and underwater cameras, inaccessible areas can be inspected and recorded. But perhaps the biggest asset of a photograph is that it can be easily and instantly understood by any one without long explanations. Statistics, charts, and bar graphs are often difficult to understand, require discipline and time to appreciate, can be manipulated and often just does not "reach" the receiver.

Thus, it was decided to use photographic techniques, wherever possible, to reproduce field conditions. Photographic elevations, almost to scale, were developed for spandrel walls and parapets, concrete arches, bascule span towers, etc. Deterioration was directly noted on these photographs eliminating necessity for majority of sketches. It is believed that this approach of using photography in engineering inspection work as well as preparing almost finished drawings has a great potential in future.

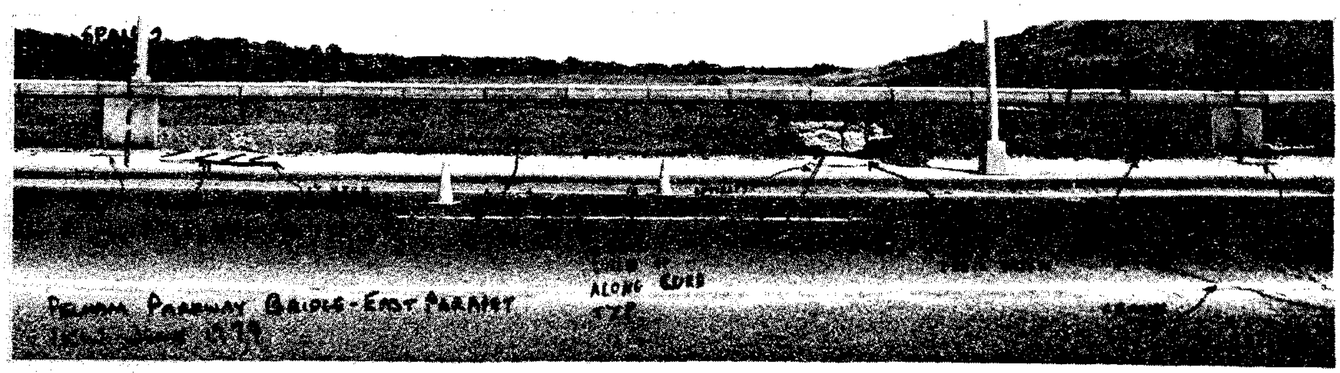
FINDINGS

The results of inspection for some of the major structural elements were as follows:

Concrete Arches

The underside of the concrete arches were sounded throughout with chipping hammers. Areas containing foreign matter, honeycombing, wetness, or cracks were examined extensively. Attempt was made to determine the depth of the faults by using cold chisels, awls, wrecking bars, and a pneumatic powered chipping gun. In some areas sound concrete was removed to determine if there was any deterioration of the encased steel trusses. Where a portion of a truss was exposed, the truss was opened up further by chipping out first bad and then good concrete to see where changes occurred in truss sections. Significant cracks with efflorescence and construction joints were also chipped out to determine the extent of the cracking or any deterioration.

The specific findings were shown on the Developed



Arch Plans. In general, these findings can be summarized to cover all spans as follows:

a. There were no diagonal or stress cracks.

b. The construction joint location varied from span to span.

c. The large longitudinal "cracks" were not cracks, but construction joints approximately midway between trusses. In chipping a "V" cut into the joint, sound concrete was usually found generally within 1 inch. The maximum depth (only at isolated areas) to sound concrete was 3 inches.

d. Honeycombing was found in different locations. Near the springline, the honeycombing was as deep as six inches.

e. In all cases where the embedded steel truss was exposed to the weather, the truss was rusted to the point where no bottom chord remained.

f. Everyplace investigated, when the truss had sufficient concrete cover, the truss had not deteriorated.

g. There were some longitudinal cracks with efflorescence along the center line of some trusses. These cracks were very tight and after chipping to a depth of one inch they faded out and did not appear related to the trusses.

h. Some areas of soffit were still wet for two or three days after a rain. Water was leaking through the sidewalk/curb/pavement cover or from the scupper/pipe system into the earth fill and then taking several days to percolate down to the top of the arch. There was no indication from the cores or original contract drawings that the arches were waterproofed. No significant amounts of porous or spongy concrete was found in the wet areas. Apparently most of the water came through the construction joints.

i. Thirty-six cores of the concrete arches were taken as indicated in sub-section, Coring Program.

j. Neither as-built drawings nor interim survey data on the arches were available. Elevations were taken of the underside of the arch crowns at each fascia of each span. Survey of the coping stones indicated that the arches have not materially settled or flattened.

Bascule Span

The moveable portion of this bascule span (trusses and bracing) was fabricated from relatively light section steel members, as was typical of the early 1900's. This resulted in a light structure but it also increased the number of connections and the exposed surface areas. The members generally are thin in section and more valnerable to rust, chemical attack and deformation. The span has an open grating deck. Structural steel members are therefore subject not only to a marine environment but are also exposed to water and road salt attack from above.

The inspection of the structural steel was seriously hampered by a 70 year paint accumulation, rust, dirt and grease, not necessarily in that order. Another factor that caused concern was the rusting patterns. When the members oxidize, they tend to turn black, remain hard, and do not flake or expand as is typical of modern steels. At one location the paint appeared sound with no visible problems; however, when chipped through several layers of paint and through hard, black, oxidized metal, the bare metal was found to exhibit a very pitted surface with craters.

The following standard procedures was utilized for inspection of the various steel members:

a. General overview.

b. Close inspection of apparently bad conditions in order to determine their extent.

c. Close inspection of areas that appear good but where trusses normally might be expected to deteriorate.

d. Inspection of random areas that appear to be satisfactory.

The findings can be summarized as follows:

a. Pins in all 4 trusses appeared round and show no evidence of wear. The westerly pin was "frozen" - the other 3 move up to 1/4" in all directions.

b. The pitting and craters seemed to occur mostly on horizontal transverse bracing. About 5% of the rivet heads had lost varying portions of their section. There seemed to be no general pattern of the steel deterioration.

c. Bottom chords of the outside trusses had apparently been hit several times by marine traffic. The horizontal angle legs had deformed locally. There were no tears or cracks at the deformation.

d. No significant deterioration, buckling, deformation, or cracking was found in any of the more highly stressed diagonal truss members nor were there any signs of overstress in their connections.

e. The horizontal members directly under the grating were rusted on the upper face. This was a difficult area to inspect. From what could be seen, the rusting is not critical to any of the members.

f. There were local areas that did need repair work. For example, the web behind the name plate on the southwest truss was completely rusted through.

g. Some "fascia girders", having been struck by marine traffic, were deformed; supports for added small counter weights were heavily rusted; horizontal transverse members (particularly those that don't drain) were rusted, pitted and cratered; and some bracing and secondary members near the counterweight needed replacement.

h. The support system for the rack and pinion and the curved track girder appeared to deflect significantly when the bascule leaf initially was moved or stopped. Many rivets were loose. No significant cracks or deformation were found.

i. It can be concluded that localized areas of deterioration exist. These areas can readily be found by sand blasting down to gray metal.

TOWERS

Span 4 (the bascule span) has four towers - one on each corner. The towers are designated as 3E for the Pier 3 easterly tower, 3W for the Pier 3 westerly tower. The exterior condition of these towers was shown in sixteen photographs - four elevations of each tower.

Unless noted, the following comments apply to all four towers:

a. The stone "caps" on each tower appeared sound but isolated joints had missing mortar.

b. Other ornamental stonework such as cap stones, coping stones, lintels, and sills, showed signs of opening joints - such as missing mortar, efflorescence under the joints and with some mortar patches under the projecting stones.

c. The towers have a veneer, probably three to four inches thick, of concrete and crushed comm n brick. With very few minor exceptions, this veneer did not appear to be delaminating in the manner of the spandrel walls and parapets. The very rough surface tended to hide defects. The upper portions of the towers tended to have small tight vertical cracks with efflorescence. The upper portion also had what appeared to be horizontal cracks but probably were weathered construction or cold.

d. The largest cracks have occurred on the north and south elevations on the far corner of of the tower away from the center line. It appeared that these cracks developed when the starlings, which were later added, settled slightly. This condition appeared stable.

e. The upper portions of the towers, from about four feet above the sidewalk, leaned toward the longitudinal bridge center line. Tower 3E was tilted the most - with the westerly face of the top dimension stone about seven inches closer to the bridge center line than the general area four feet above the sidewalk. Some of the ornamental stonework was either broken or worn off over the years when the bridge was raised into the completely "up" position. To avoid any further damage to the steel/concrete/stone work, the operators have apparently been instructed not to raise the bridge into the completely "up" position. The other three towers did not lean as much, but they were similarly tilted about three inches.

f. According to the operators, under the right combination of wind and rain, all four towers had leaking "roofs". There was ample evidence of very crude caulking/covering of joints and interfacings. The sloped roofs had raised seam copper roofing. Gutters and leaders were either missing or leak so badly as to be ineffective. All exposed wood in the eave area was unpainted and often rotted.

In general, the four towers appeared to be functioning as they were designed.

EVALUATION

Based upon the findings of the In-Depth Inspection, Stress Analysis and Rating, Performance Observations and Tests the following was concluded:

1. Rehabilitation of the seventy year old Pelham Parkway Bridge on Eastchester Bay was feasible and practical.

2. All bridge elements can readily be upgraded to support all of the following AASHTO live loadings: H20 - 44, HS20 - 44, Type 3, Type 3 - 3, and Type 3 S 2.

3. The six basic concrete arches were reasonably sound and after removal of roadway pavement, fill, spandrel walls, etc., can be rehabilitated and then "capped" with a continuous strip of reinforced concrete. Following introduction of a waterproofing membrane in conjunction with new spandrel wall construction; replacement fill, road and sidewalk pavements, drainage system, etc., can be installed. A net allowable working stress of 600 psi can be used for the existing concrete arch construction.

4. Structural steel elements of the bascule construction were were reasonably sound and following a cleaning, strengthening/replacement, and painting program can be continued in service.

5. Tower, pier, and abutment structures were adequate and can simply be rehabilitated

and continued in service.

6. The fender system adjacent to the channel was in generally poor condition and should be replaced.

7. Modifications could and should be made to the existing utilities - lighting, Communication, water, sanitary - to bring them up to current standards and operating efficiency.

8. Introduction or upgrading of a number of features relating to pedestrain, vehicular, operational, and marine safety to insure compliance with modern standards were required and feasible.

9. It was not feasible to widen the existing four-ten foot lanes, or introduce shoulders, or provide a median barrier between opposing lanes of traffic.

CONSTRUCTION COST ESTIMATES

A detailed cost estimate was prepared for the rehabilitation of the structure based upon evaluations discussed earlier and retaining the existing geometry.

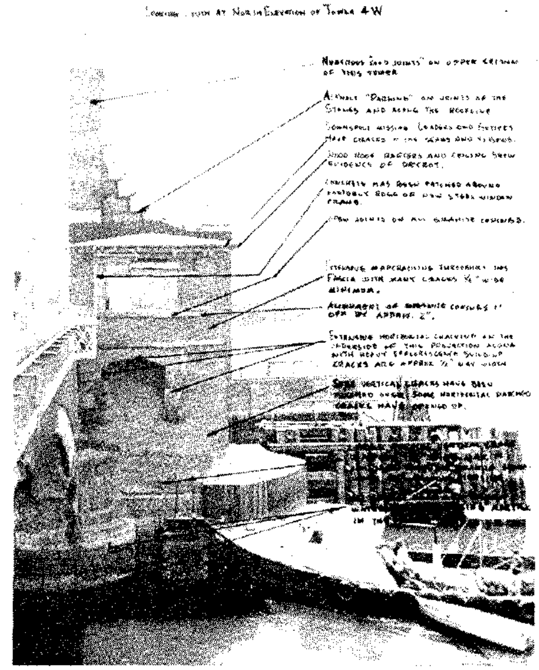
No attempt was made to prepare an estimate for rehabilitation to include widening of the structure to meet current geometric design standards. Because the complexities involved in widening the existing arches and substructure, removing and rebuilding the towers and control houses as well as replacing the existing bascule span with a completely new one made this an unfeasible alternate with a cost approaching that of a new replacement structure for what would remain essentially a seventy-five year old structure.

A detailed cost estimate was also prepared for a new structure adjacent to the existing structure and meeting current geometric standards.

The cost of rehabilitation was estimated to be about half the cost of the new structure.

AKNOWLEDGEMENTS

The author wishes to acknowledge the valuable assistance provided by the clients of this project; New York City Department of Transportations.



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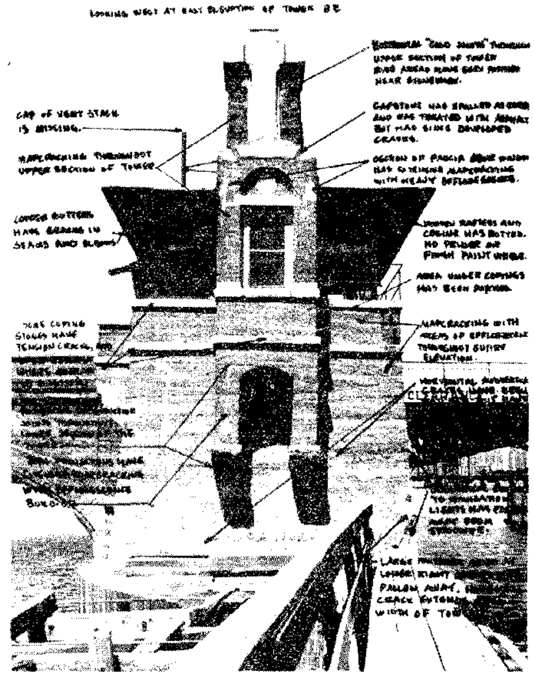
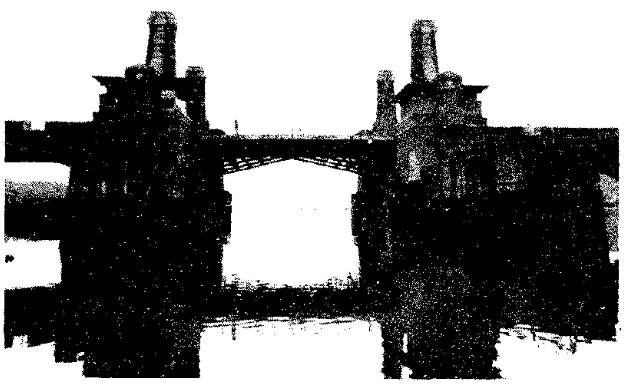


ELEVATION OF TOWER S.E.



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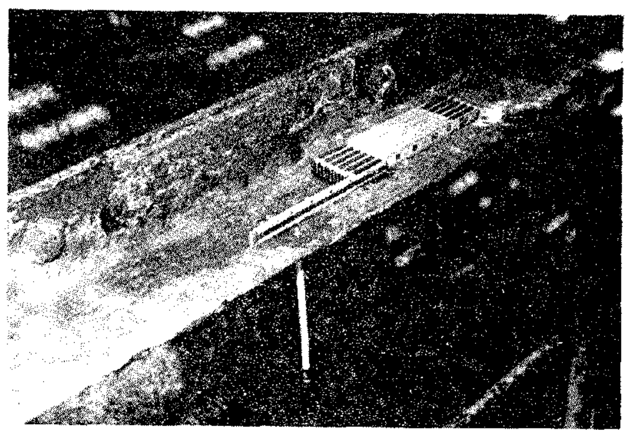
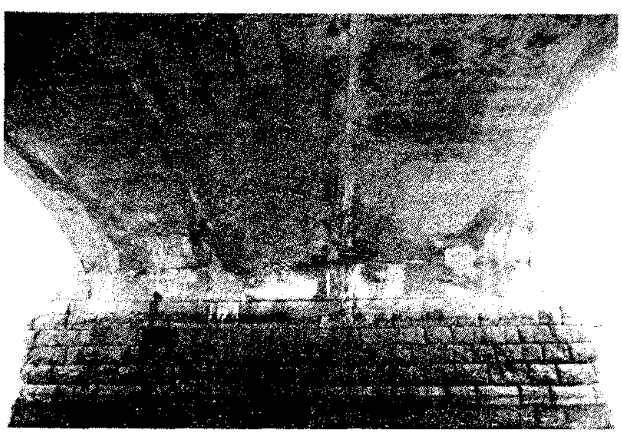


DETERIORATION OF BOTTOM CHORD ANGLE



UNDERSIDE OF ARCH, SPAN

DETERIORATION OF RIB BOTTOM CHORD

USE OF PRECAST CONCRETE COMPONENTS FOR REPLACEMENT AND REHABILITATION OF BRIDGES

Leslie D. Martin and Norman L. Scott
The Consulting Engineers Group, Inc.
Glenview, Illinois USA

INTRODUCTION

Possibly the most pressing need in highway construction today is the repair or replacement of bridges which are structurally deficient or functionally obsolete.

The extent of deficiency to which the results of this study may apply can cover a broad range:

.....Deck surface deterioration requiring the replacement of the upper portion of the deck

.....Deck deterioration or obsolescence which requires the complete replacement of the remainder of the superstructure. This would include, for example, a combined need for deck replacement and widening

.....Deterioration or obsolescence of the whole superstructure, but pier and abutments remain usable.

.....Deterioration or obsolescence which requires complete replacement

Previous research and experience has shown that modular precast concrete construction may be applicable to all of these repair or replacement requirements. Of course, modular construction is also used for new construction.

It is apparent that when a bridge is closed for repair or replacement there is a serious economic impact as well as major inconvenience to the users. Prefabrication of components can significantly reduce the out-of-service time for bridge repair or replacement. This time reduction can also reduce costs that result from inflation escalations. Cost savings may also be realized because of the reduced field labor employed. Increased standardization of components, details and procedures can result in further time and cost reductions.

Precast concrete is currently being used in three different methods of construction on bridges: 1) precast, usually prestressed members which span between girders and act as "stay-in-place" forms to receive a cast-in-place deck; 2) precast concrete units used as a full precast deck on steel or prestressed concrete bridge girders; and 3) precast units, usually prestressed, which have integral decks and girders.

STAY-IN-PLACE FORMS

Description

In this type of construction, thin precast, prestressed concrete members are placed between the prestressed concrete or steel girders. A cast-in-place, reinforced concrete deck is then poured on top of these members (See Fig. 1). In most cases the precast members are designed to act integrally with the cast-in-place concrete and compositely with the girders. Numerous uses of these slabs have been reported in the literature, and the Prestressed Concrete Institute has published design and construction specifications. Several variations of the slabs have been used. In most cases the top surface is intentionally roughened (to varying degrees). Sometimes metal ties, reinforcing bars, wire, etc. are projected from the surface of the precast slab. In other cases a machine made slab is used that has transverse (to direction of traffic) ribs as shown in Fig. 2. In the machine made sections strand normally does not project from the end of the member. Some state specifications require a reinforcing bar to be placed between adjacent slabs when the strand does not project, as shown in Fig. 3.

Past and Present Usage

Illinois Tollway

The use of precast prestressed concrete deck panels to act as stay-in-place forms for highway bridges was first proposed in the mid 1950's for the Northern Illinois Toll Highway. In order to evaluate the performance of this type of structure, a representative bridge was erected in advance of the rest of the Tollway, and load tests were conducted at various stages of construction.

With the exception of the cast-in-place slab over the deck panels, the entire bridge utilized precast construction techniques. The precast panels were 2-1/2" (64 mm) thick and featured tongue-and-groove lateral panel joints. These joints proved to be subject to damage during construction and are no longer used. The slab was roughened, but no reinforcement was projected from the surface, other than that required for handling the slabs. A 4-1/2" (114 mm) topping was then cast directly over the precast panels.

Testing procedures involved placing a series of static loads on the bridge, as well as dynamic impact loadings provided by a fully loaded earth mover. Two precast panels with a 4-1/2" (114 mm) topping were tested to failure in the laboratory. At no time during the testing did separation between the cast-in-place topping and the precast panel take place, even at failure.

A comparison of the observed strains in the bridge with predicted strains for an equivalent cast-in-place bridge confirmed that complete and positive composite action between the precast girders, precast slabs and cast-in-place slab is attained.

As a result of these full scale tests, this precast deck panel system was used on several other bridges on the Northwest Tollway and, later, on the East-West Illinois Tollway.

Observations in the spring of 1980 showed no problems in the performance of the bridge. The asphaltic concrete surface had one or two pot holes normally associated with the material. This appeared to have no effect on the deck.

Texas Highway Department

In 1963 the Texas Highway Department constructed three bridges similar to those built in Illinois. The Texas bridge deck panels were 3-1/2 to 4 in. (89 mm to 102 mm) thick and they rested on fiberboard strips (instead of a grouted bed, as in Illinois). Lateral joints were butt-edged, allowing for a simpler casting procedure.

Observations after seven years of service revealed no evidence of distress or non-composite action in any of the three bridges. Some transverse cracking was found to coincide with the transverse butt joints between prestressed panels.

Subsequent full-scale laboratory tests of a single span bridge of this type proved composite behavior and satisfactory transfer of wheel loads across panel joints.

Pennsylvania State University

The most comprehensive full scale load test of a bridge of this type took place at Pennsylvania State University's pavement research facility from 1972 to 1977. A two-span prestressed concrete I-beam bridge was constructed using two different types of deck systems. One span was built with conventional forms, while the other span was formed using precast, prestressed stay-in-place panels. A 4-1/2" (114 mm) thick topping slab was placed on the 4 ft (1.2 m) wide by 3 in. (76 mm) thick panels. Two types of panel-to-panel joints were incorporated in the deck (See Fig. 4).

Testing procedures involved driving a semi-trailer loaded with steel ingots across the bridge for 500,000 and 1,100,000 cycles of equivalent 18 kip (80 kN) axle loads. Upon completion of these service load tests, the bridge was tested for overload using a trailer capable of imposing 60 kips (267 kN) on each wheel. No cracking was observed in the deck until application of the full 60 kip (267 kN) wheel loads on tandem axles spaced 4 feet (1.2 m) apart, at which point, diagonal tension cracks appeared in the deck.

As in the previous studies, the precast panel was found to act fully composite with the concrete topping for all load cases, including overload to failure. The following conclusions were among those drawn from this testing program:

1. The joints between the plank forms do not affect the longitudinal wheel load distribution in the deck.

2. The type of joint used between deck plank forms has no effect on the behavior of the deck.

3. Mechanical shear connectors are not required if the plank surface is given a scored finish.

4. The absence of intermediate diaphragms did not adversely affect the behavior of the bridge under load.

Virginia

A similar experimental bridge was constructed in Virginia in 1977. After one year, hairline cracking was observed on some of the decks above the joints between panels. While similar cracking is often found in conventionally constructed decks, the joints between the subdeck panels appear to control the location.

The report recommended that this method be considered a viable alternative to conventional deck construction.

Others

The use of precast, prestressed stay-in-place forms has spread widely over the United States. The 1979 AASHTO Interim Specifications for bridges, Section 2.4.3(P) covers construction of such decks. In most places where this method is used it is allowed as an alternate to conventional forming for the deck. Most states specify a section similar to that recommended by the PCI "Tentative Specifications", and at least one state (Louisiana) allows two alternates to conventional cast-in-place concrete, both roughened surface types and machine-made ribbed sections (Fig. 2).

Nearly all applications to date have been with prestressed concrete girders in new construction. One project near New Orleans used stay-in-place forms with steel girders, and Tennessee DOT has used precast forms with steel girders for four years. The New York Department of Transportation has used a precast nonprestressed section to replace a deteriorated deck on steel. Other states have established standards for deck forms on steel.

Evaluation & Conclusions

The stay-in-place form as a practical and economical means of construction of bridge decks appears to have proven itself, both in terms of longevity, considering the Illinois Tollway Bridge, and frequency of use. There has also been enough controlled research on this subject to verify that the system will perform adequately on high volume, high traffic bridges and will withstand significant overloads adequately.

The only problems that have been reported were minor cracking in the deck surface. In most cases, the cracks appeared to coincide with the joints between panels, or longitudinally along the girders at the ends of the panels. Since this cracking was sometimes noticed before the bridge was open to traffic, it seems likely that it is a result of shrinkage of the cast-in-place deck, rather than flexure. At any rate, it was generally considered no more severe than normally found in decks formed conventionally.

The Illinois Tollway tests showed that a 2-1/2 in. (64 mm) precast slab was adequate. This would be more economical than the 3 to 4 in. (76 to 102 mm) slabs used elsewhere. Furthermore, the joints in the precast slabs act as contraction joints. By reducing the ratio of precast to cast-in-place slab thickness, it is likely that deck cracking would be reduced.

The Pennsylvania State University Research Program resulted in an "Implementation Statement" by the researchers for the Pennsylvania Department of Transportation. With modifications discussed above, this statement can serve as a model guide specification for the use of this construction method.

FULL DEPTH PRECAST CONCRETE DECKS ON GIRDERS

Description

In this type of construction, the entire bridge deck is of precast concrete. There is no additional field cast concrete that acts structurally, except that which may be used in the connections. There may be a riding surface of asphaltic concrete, or in some cases, high density portland cement concrete or other material applied for the purpose of combating deterioration.

The deck may or may not be designed to act compositely with the girder, depending on the connection of the deck to the girder and the confidence of the designer.

Past and Present Usage

Most applications of full depth decks on girder bridges have been for replacement of deteriorated concrete decks on steel girders. Even these applications have been relatively few, and typically in situations where rapid reopening to traffic was important.

Several of the projects described in this section relate to research and others were considered "experimental" at the time they were placed.

Purdue University - Indiana State Highway Commission

In 1969 a research project at Purdue University was conducted to determine the feasibility of using precast, prestressed concrete deck members on steel beams. The research consisted of comprehensive prototype testing in the laboratory and two bridges constructed by the Indiana State Highway Commission.

The test bridge deck consisted of narrow precast, pretensioned planks placed transversely to the traffic flow. The planks were connected together with a tongue and groove joint, and post-tensioned longitudinally, as illustrated schematically in Fig. 5.

A photoelastic investigation was made of the three different joint shapes shown in Fig. 6. This study indicated that the flat joint (Fig. 6a) was superior to the other two. Laboratory cast specimens using this joint were post-tensioned to 40 psi (276 kPa) and tested with repetitive loads through 2.25 million cycles without distress.

The successful laboratory testing resulted in two experimental decks being constructed on the Indiana Highway System.

One of these experimental bridges is on Indiana State Road 140 over the Big Blue River just south of Knightstown. The structure is a new, three span continuous steel beam bridge having spans of 70 ft-70 ft-60 ft (21-21-18 m), Fig. 7. The deck slabs are 38 ft-4 in. (12 m) long, nominally 4 ft (1.2 m) wide, positioned on steel beams spaced at 6 ft (1.83 m) on center. The slabs have a built in crown with the thickness of each slab varying from seven inches (180 mm) at each end to 10-1/2 in. (270 mm) at the center (See Fig. 8). To reduce stress concentrating at the joint, a 1/16 in. (1.6 mm) V60 neoprene sheet was placed in the joint. The sheet material was also intended to minimize water leakage through the joint (See Fig. 9) Fig. 10 shows the bolted clip tie-down connection.

An inspection of this bridge by the researchers in June, 1974 indicated that the main problem was due to irregularities in the width of the joints at the top of the slab. After the slabs were post-tensioned together, there were numerous locations where the joint widths were less than 1/8 in. (3 mm), and approximately half of the defective joints were completely closed. There were no immediate effects except the joints leaked water when it rained because no sealant could be installed in the closed joint. However, a few months after the bridge was opened, the concrete in the vicinity of the closed joints began to spall. In some areas the spall has been large enough to require an asphalt patch. The cause of the joint irregularities was irregularity in the forms used to cast these slabs.

A May, 1980 inspection also noted that in many cases the joint sealant did not effectively bond to the edges of the slabs and joint leakage is a continuing problem. Many of the beam clips showed signs of severe corrosion.

New York Thruway Projects

The New York State Thruway authority has used full-depth precast concrete decks to replace deteriorated concrete at three different locations. The first one was at Amsterdam, New York and was set up as an experimental project. This bridge was constructed during the autumn of 1973 and the spring of 1974.

The objective of this prototype project was to evaluate the effectiveness of both welded and bolted connections, each designed to accomplish composite action with the steel girders. While this is a four span bridge, precast panels were placed on only one-half of span 2, with the remainder repaired using conventional methods (i.e., cast-in-place concrete on removable wood forms). A total of seven panels were placed in each of the lanes. Three of them were connected with a bolted connection as shown in Fig. 11a. The other four used a welded connection as shown in Fig. 11b.

The first panel placed used the bolted connection. Steel shims were used as required to adjust for slight elevation differences in the top flanges. Temporary steel-to-concrete connections were made by screwing clips into cast-in-place threaded anchors in the panels. The bolt holes in the top flange of the steel girder were drilled through the sleeves in the precast unit. High strength bolts were then placed and turned to the specified torque.

The remaining four slabs installed used the welded connections shown in Fig. 11b. An epoxy gel mortar (two parts gel to one part sand) was trowelled on the stringers for the first two welded panels. The gel was used for leveling in lieu of the metal shims. The adhesive strength between the stringer and panel was neglected in the design calculations. The next panels were installed on an epoxy mortar of one part resin to 2-1/2 parts aggregate, and poured on the stringer in a wide bead.

This project was visited in May of 1980, more than six years after it was constructed. There was no evidence of any difference in performance between any of the three types of construction used on this project: bolted panels, welded panels, or conventional cast-in-place deck.

The deck had been water-proofed with a sheet membrane and overlayed with asphaltic concrete in two courses. There was no evidence of moisture leakage through the deck.

The second bridge constructed using this type of precast panel was on the thruway itself where it crosses Krum Kill Road near Albany. This is a single span overpass and precast panels were used on the entire crossing. Connections similar to that used in the welded portion of the Amsterdam bridge were specified for this bridge, with an alternate using welded headed studs instead of the steel channel weldment for the horizontal shear transfer. The welded stud alternate was chosen by the contractor. Figs. 12a thru 12c show construction procedures used on the Krum Kill Bridge. Fig. 12a shows the preparation of the steel prior to placement of the slabs. An epoxy mortar bed is placed on the steel girder prior to placing the concrete slabs as shown in Fig. 12b. The slabs are then lifted into place (Fig. 12c), and after that the studs are welded through the openings in the deck to the steel girders.

The bridge performance has been satisfactory. There was no evidence of reflected cracking through the asphaltic concrete surface and there is no evidence of moisture leakage through the slab.

The third application of this method of construction was the entrance ramp to the thruway in Harriman, New York. Since this was a curved, super-elevated bridge the precast planks would not fit level on the beam flanges. Therefore the epoxy mortar bed was thicker on one edge of the flange than it was on the other. This did not present any particular problems in construction nor is there evidence of impairment in performance.

State of New York

The New York State Department of Transportation has replaced decks on several steel bridges with precast concrete planks. The first of these was a 1,040 ft (317 m) long suspension bridge over Rondout Creek near Kingston, New York. the precast deck was chosen for this bridge because of the need to load and unload the suspension bridge in an ordered sequence, coupled with the need for speed.

The deck slabs are 9 ft (2.7 m) wide and 24 ft (7.3 m) long with a simple V male-female joint, with no grouting or caulking except at the connections to the steel stringers. The slabs are bolted together longitudinally with tie rods. Details are shown in Fig. 13.

After five years of operation the Kingston Bridge had shown no apparent deck problems. Initially there was some pushing of the asphalt overlay on one end of the bridge and, because of its narrow width and heavy traffic, there was some rutting of the overlay.

A three span structure over the Delaware River between Sullivan County, New York and Wayne County, Pennsylvania was opened in 1978. The total length is 675 ft (206 m). The panels are 7'-6" (2.3 m) by 13'-10-1/2" (4.2 m) joined together transversely and longitudinally at the centerline by epoxy shear keys. They are attached to the steel stringers by a single stud at two points on each panel as shown in Fig. 14.

A third bridge is in southern Erie County, New York and is also a three span bridge, 540 ft (165 m) in length. It was built in 1979. The panels are 8 ft (2.4 m) wide and about 22 ft (6.7 m) long, with transverse and longitudinal centerline joints identical to the Delaware River bridge. The attachment to the stringers is somewhat different, as shown in Fig. 15.

All of these installations are reported to be performing satisfactorily, although there is some evidence of leakage at some joints.

Santa Fe Railroad Bridges

Several old timber decks, many of them 50 to 60 years old, have been replaced with precast, prestressed concrete decks in recent years on the Santa Fe Railroad. These decks bear on riveted plate girders, most of which were erected in the late 1890's or the early 1900's. Initial construction had open decks that were converted to ballasted decks about 20 years later. The use of ballasted deck construction is highly desirable from a track maintenance standpoint, so it was decided that the replacement of these bridge decks would also include the replacement of track ballast and standard ties.

After removing the track ballast and the old timber deck, the top flanges of the steel girders are sandblasted clean. Then narrow plywood screeds are bonded along each edge of the cover plate using an epoxy gel. The thickness of these screeds is equal to the height of the rivet heads projecting above the cover plate (See Fig. 16). A layer of sanded epoxy grout is then poured over the entire top cover-plated surface, slightly overfilling the screeds so that it is extruded by the weight of the slab, assuring a fully bonded contact surface. The slabs are eight feet (2.4 m) wide (parallel to the track) by 14 ft (4.3 m) by 8 in. (200 mm) thick and are obtained from a commercial prestressing plant located on the Santa Fe line in Albuquerque, New Mexico. The slabs are cast upside down so that the dense steel formed surface will be on top after erection for maximum water resistance, and the rough broomed surface will be on the bottom to give the highest possible bond with the epoxy grout placed on the top flanges of the girders.

Treated timber ballast curbs are then bolted in place. Deck anchors with spring compression clips are placed at 4 ft (1.2 m) on center along the girder flanges to provide lateral strength if the epoxy bond between steel and concrete should ever break down. Track panels are replaced on blocking, ballast replaced and the traffic is restored. Production has ranged from 24 to 48 track feet (7.3 to 14.6 m) of deck replaced per day. Work is tightly scheduled between passing trains to avoid disruption of traffic.

Although the only positive connection between the precast deck and the steel girders is an epoxy grout, the bridges are designed for composite action. Stress tests with SR4 strain gages before and after the deck renewal verified composite behavior. This composite action increased the strength to the extent that renewal of the corroded cover plates will not be required.

Interstate 80 Overpass, Oakland, California

This project is a good example of replacing a deteriorated deck while maintaining traffic on a busy urban freeway (See Fig. 17). From 60 to 80 feet (18 to 24 m) of old concrete, 12 feet (3.7 m) wide, was removed each day, leaving the girders bare. The new panels, 30 to 40 feet (9 to 12 m) long and weighing up to 18 tons (160 kN) were cast near the site and trucked to the installation areas. Oblong holes 12 inches (300 mm) long and 4 inches (100 mm) wide, were formed in the panels. Four shear connector studs were welded to the girders through each hole. The studs were 7/8" (22 mm) in diameter and 6" (150 mm) long.

Two unusual procedures facilitated final attachment of the panels to the girders. The first was the use of two-headed bolts, 1" (25 mm) diameter, that were turned into threaded sockets cast in the panels. By turning the bolt heads on top of the panels, the panels were raised or lowered until level. From 1" to 2" (25 to 50 mm) of "air" was left between panels and girders.

After panels were in place, wooden forming was placed vertically beside the girders underneath the panels. Final step was pouring fast-setting concrete into the holes where studs were welded. The concrete filled the holes and also flowed along the girders between the forming.

During each 24 hours, work started at 8:00 P.M. when the lane being worked on and one other lane were blocked off. At 7:00 A.M., the untouched lane was reopened. At 2:30 P.M., the "new" lane was also reopened for 5-1/2 hours of the day's heaviest traffic.

Clark's Summit Bridge on the Pennsylvania Turnpike

Precast panels were chosen for the replacement of the deteriorated deck on this bridge shown in Fig. 18. They were chosen because it was necessary to maintain traffic on half the bridge while redecking the other half. It was feared that vibrations from the traffic could interfere with the proper concrete setting, especially at the juncture of the new decks. Other reasons included: 1) Safety - the Clark's Summit bridge passes over two highways and one railroad at about 140 ft (43 m) elevation at its highest point. No deck forms were required, thus eliminating much of the dangerous field work normally involved with installing forms for cast-in-place concrete 2) Time - It was estimated that the time for replacing the deck with precast slabs would be from 1/2 to 1/3 of that casting a conventional deck 3) Cost - At 2.8 million dollars the cost, including other substructure repairs, was comparable to the cost of the cast-in-place deck.

Connection details for the slabs are shown in Fig. 19. After removal of the old deck the top flanges of the existing steel beams and girders were sandblasted. The top flanges were painted with a coat of epoxy to receive neoprene strips. Epoxy grout with a light overfill was placed between the strips as a levelling course. The inserts and bolts shown in Fig. 19 were placed near the edge of the slabs so they could be tightened by reaching under the slabs from the top. Slabs were bolted longitudinally and a non-shrink grout was placed in the shear key between the slabs.

These bridge slabs were not designed for composite action with the girders although it is likely that some composite behavior results with this detail.

CERL - Champaign, Illinois

The U. S. Army Research Laboratory in Champaign, Illinois is currently conducting a related study on a segmental polymer-impregnated prestressed concrete bridge deck concept for the Federal Highway Administration. The study involves design and structural testing of the full-depth deck system. The waffle-shaped panels are connected to the supporting steel girders with welded headed studs in an epoxy grout pocket. The panels are set on an epoxy grout bed on the beam flanges. Joints between the slabs are filled with epoxy mortar. Static and fatigue loading indicate full composite action between panels and supporting girders.

Summary & Evaluation of Connections

The type of construction described in this section involves two types of connections: 1) connection of the deck to the girders, and 2) the connection of adjacent panels.

Connections of Deck Slabs to Girders

Of the several projects using precast concrete slabs on girders, some considered the deck and supporting beam to act compositely and some were designed non-compositely. The available research seems to indicate that it is entirely possible to get composite action, and there seem to be very few reasons not to use it.

The New York Thruway projects are examples of assuring composite action. These projects used both an epoxy coating on the girders and a mechanical tie between the girders and the slabs through a block-out. In this case the design was based on the mechanical connector providing the full horizontal shear tie between the slabs and the girders. The epoxy mortar over the girders was primarily used as a leveling device, although it was recognized that some adhesion was present and that this would provide additional safety for assuring composite action. The Corps of Engineers' test showed that composite action is definitely obtained with this type of connection under static and dynamic loading.

The Oakland Bridge used only standard sand cement mortar between the slab and the beams. The purpose of this mortar was merely to provide vertical load bearing capacity. Leveling was done by other means. Composite behavior was established by the welded studs in grout pockets. Clark's Summit Bridge used epoxy mortar that was not designed compositely.

Conversely, the Santa Fe Railroad Bridges used epoxy mortar without any mechanical ties except for spring clips, which cannot be counted upon for significant shear resistance. Their field tests indicated they were getting satisfactory composite action with just the epoxy bonding , but the limited nature of the tests may not satisfy most bridge engineers. The Purdue-Indiana Highway Commission test bridges also seemed to verify that epoxy bonding will provide composite behavior.

In every case we investigated , either through personal inspection, reviewing the literature, or discussions with bridge engineers, it appeared that every connection tried worked satisfactorily for its intended purpose. In most cases, it would seem that the use of both epoxy mortar and mechanical ties is unnecessarily conservative. However, controlled testing of the mechanical ties would give additional assurance to some design engineers. It should be noted that standard composite design calls for mechanical ties, usually headed steel studs, spaced uniformly along the steel girder. The type of connection studied here involves the mechanical ties being spaced somewhat further apart and could result in some skepticism on the part of the designer.

In many of the projects investigated, the epoxy mortar was used for leveling the slabs. The double-headed bolts used on the Oakland project would seem to be more positive and faster. At Oakland, the space between the slab and the girder flange was formed from below and a flowable grout poured into the connector blockouts. This method of forming would seem to be costly and dangerous, and use of plywood strips, as done on the Santa Fe Railroad jobs, or neoprene strips, as used at Clark's Summit, would seem preferable. A neoprene tube or O-ring or polyethylene foam rod could be used for this purpose. Its advantage is that it is more compressible and hence would allow more tolerance than plywood or solid neoprene. The disadvantage of any of these methods compared with forming from below is that it is one more operation that must be performed prior to placing the slabs. On the tight schedule necessary on the Oakland bridge this may have been a consideration.

Connections with Adjacent Slabs

This connection serves two functions. First, to provide lateral load transfer across the joint, and second, to prevent moisture leakage through the slab. Several of the projects inspected used standard grout keys between the slabs and either an epoxy mortar or a sand-cement grout in the joint. With the limited experience available it appears that either of these methods works satisfactorily. However, the sand-cement grout joint, which is usually used more extensively in the integral deck bridges may not hold up in very high volume traffic.

The "dry" joint used on the Indiana bridges was not entirely satisfactory on either count. The joint configuration chosen was too sensitive to forming tolerances, causing stress concentrations and resulting in concrete spalling. The simpler V-joint used on the Kingston, New York bridge may have worked better, but since it was covered with asphalt, it could not be determined if it did. The Indiana joint obviously fell far short of preventing moisture leakage. It appears that dry joints (no cast-in-place concrete or grout) would only be applicable with some additional waterproof membrane and surfacing, if moisture leakage is a performance problem.

Inhibitors for Extended Use

The biggest problem with this type of construction at this date would seem to be its cost. The Erie County Bridge in New York State was estimated to cost about 50% more than a cast-in-place monolithic deck. Few other direct cost comparisons were made.

In most of the projects reported in Sect. 3.2 the precast deck units were chosen because of anticipated increase in speed of construction. In those cases it was necessary to get the road back in service as quickly as possible, and it was felt that the precast construction would do that job. Several users have reported disappointment in the time savings. The Oakland Interstate bridge, and the Santa Fe Railroad experience illustrate that with proper planning, the anticipated time savings can be realized. For the Clark's Summit bridge, precast planks were chosen because it was feared that the vibrations from traffic on the adjacent slab during construction would interfere with proper setting of the concrete in the span under construction. For the Kingston bridge in New York state precast slabs were chosen because of a need to load the suspension bridge in a proper sequence. In several of the projects the use of the precast panels was partially justified on the basis of providing higher quality concrete, which is anticipated to inhibit deterioration of the deck.

It would seem that one of the primary reasons that the cost of using precast panels on bridge decks was so high is that in nearly every case, the method was considered "new". Standard methods of construction and design for this method have not been developed and contractors have not been able to gain repeat experience. It would seem that as long as this is used only for replacement of deteriorated decks this will continue to be the case. Some cost reduction in the process can be attained with less costly connection details. However, to convince most design engineers that a "belt and suspenders" approach to design is not necessary would require some testing of the connection details.

INTEGRAL DECK BRIDGES

Description

This category of precast concrete bridge decks includes those types in which the deck and the main supporting members are the same (such as a slab) or those in which the deck and the structural supporting unit (the beam or girder) are cast as a single unit. In some cases a cast-in-place composite deck is poured on top of these units. In others there is merely a wearing course or in some cases no additional treatment at all.

Types of Sections Used

A wide variety of sections have been used for integral deck bridges, these are shown in Fig. 20 and are listed below:

a) Solid and voided slabs - Standard sections in the United States include solid slabs 12 in. (300 mm) deep and either 36 or 48 in. (914 or 1220 mm) wide and voided slabs that are 15 in. (380 mm), 18 in. (460 mm), and 21 in. (530 mm) deep and either 36 or 48 in. (914 or 1220 mm) wide. These slabs are normally used for spans of less than 35 ft (10.7 m). The voided slab sections mentioned above can span up to 50 ft (15.2 m) with HS 20 loading.

b) Box beams - Standard box beam sections have also been developed for 36 in. (914 mm) and 48 in. (1220 mm) wide, depths of 27 in. (690 mm), 33 in. (840 mm), 39 in. (990 mm), and 42 in. (1070 mm). In many cases these box beams are used spread from 3 ft (0.91 m) to 4 ft (1.2 m) apart with a cast-in-place deck spanning between them. For the deeper sections, spans in excess of 100 ft (30.5 m) are possible with HS 20 loading. Box beams for railroad loadings have also been standardized.

c) Channels - Precast concrete channel sections have been used for bridges for over thirty years. During the early years the sections were mild steel reinforced but now most are prestressed. Channels are usually made in depths of 21 to 35 in. (530 to 890 mm) and widths from 40 to 66 in. (1020 to 1680 mm). Spans are usually from 20 to 60 ft (6.1 to 18.3 m) without a composite cast-in-place deck, and up to 70 ft (21.3 m) with a composite deck.

d) Double tees - This section has been used mostly on county, municipal or private roads, although a few state highway departments have used them (Fig. 20). Many of these bridges use standard thin stemmed sections up to 36 in. (910 mm) deep, while others use a wide stemmed member developed especially for bridge use. Spans up to about 60 ft (18.3 m) and more with HS 20 loading are possible.

e) Single tees - The standard single tee or "Lin tee" developed for long spans in buildings has been used extensively for bridges (Fig. 20). Flanges are usually thickened for bridge use and may be cut back from the standard 8 ft (2.4 m) width to 4 or 6 ft (1.2 or 1.8 m), unless a cast-in-place topping is placed on top. Depths from 24 to 48 in. (610 to 1220 mm) are available in some parts of the country and these units can span up to 120 ft (36.6 m) in the four foot (1.2 m) width.

f) Bulb tees - A special bridge section called a bulb tee has been used extensively in the Pacific Northwest and Alaska (Fig. 20). Sections with depths of 29 to 77 in. (740 to 1960 mm) have been used and spans of up to 180 ft (54.9 m) are possible in the very deep sections. Because of the high section modulus to weight ratio this section is gaining acceptance for a wide variety of bridge uses.

g) Multi-stemmed units - A relative newcomer to the short span bridge market is the multi-stemmed tee which is especially suitable for spans of 25 to 55 ft (7.6 to 16.8 m). Depths range from about 16 in. (400 mm) to about 24 in. (610 mm) and widths are typically at 4 ft (1.2 m). The most extensively used section has four stems as shown in Fig. 20, but other sections with 3 stems or in one case 5 stems (this section is 6 ft (1.4 m) wide) have also been used.

Types of Deck Connections

Sand Cement Grout Keys

By far the most common method of connecting adjacent integral deck units is with the use of a key filled with grout (See Fig. 21). In some cases a non-shrink component (aluminum powder) is added to the grout, or a commercial non-shrink grout mix is used. In other cases, only sand-cement and water are the grout components. These joints typically require some sort of mechanical tie between the units laterally, to assure that there will be no transverse displacement. Various types of transverse ties are described below.

Transverse Bolts or Tie Rods

The most common method of tying box beams, slabs, and sometimes channels together transversely is with bolts or threaded rods which are tightened to a specified torque. This is illustrated in Fig. 22. Very often these transverse bolts are placed in the diaphragms, although in the case of channel slabs short length bolts, simply tie adjacent stems together at quarter points.

Transverse Post-tensioning

After experiencing some cases of poor grout key performance when transverse bolts or tie rods were used, some states and some contractors doing private work have used transverse post-tensioning. The post-tensioning used consists of either strands or bars and is thought to give a more positive lateral tie. In the case of box girders these post-tensioning tendons are nearly always passed through sleeves in the diaphragms and grouted.

Weld Plates

In the Pacific Northwest and other places where stemmed units are used, the most common type of transverse connection is a weld plate placed in the flanges of the units. These weld plates are spaced from 4 to 6 ft (1.2 to 1.8 m) apart along the length of the member. Size of the plate varies as does the connection detail into the flange of the member. Typical weld plate details which have been used are shown in Fig. 23.

Epoxy Grout Key

The state of Florida specifies an epoxy mortar for the grout keys on slab bridges. The Southern Pacific and the Santa Fe Railroads have used epoxy grout keys for box beam bridges. The keys are sandblasted in the precasting plant prior to shipment of the units. After the prestressed members are in place the grout key is filled with a well-graded, small-sized aggregate, typically a pea gravel. A low viscosity epoxy gel is then poured into the joint. No transverse mechanical ties are used on these bridges and performance has been reported as satisfactory.

Summary and Analysis of Integral Deck Bridge Connections

Longitudinal Connections Between Members

Integral deck precast, prestressed concrete bridges have attained over 25 years of generally excellent performance. In those cases where four or more inches of cast-in-place composite topping is used, there have been no significant connection problems reported. Thus it can be stated with some confidence that the best connection is a thick layer of cast-in-place reinforced concrete.

This type of deck, however, is more costly and adds significantly to the construction time. Also, when deicing salts are used, the same problems are encountered as when concrete decks are used on other types of supports.

The vast majority of the bridges which do not have a cast-in-place topping use grout keys to connect adjacent units together. There have been numerous cases of unsatisfactory performance of such shear keys, and a few reports of complete failure of the keys.

In this study, alternatives to grout keys for these longitudinal joints were investigated. One possibility is a tongue-and-groove joint. The experience with the Purdue University-Indiana Highway Commission experimental project points up the difficulties with this method especially in relatively thin, 6 in. (150 mm) or less, deck sections. On the test bridges, they found that an exact fit is nearly impossible without match-casting. For flat slabs, match-casting may be feasible, but it certainly is not practical for the other types of sections used, since most are cast in plants equipped with long-line pretensioning beds. Match cast flat sections would require relatively short length beds at least as wide as the bridge.

The final report on the Purdue study suggested that if an asphaltic concrete surface is used, many of the problems experienced could be eliminated. The Kingston, N. Y. deck replacement would seem to bear this out. However, it is probable that costs would be at least as high. The tolerance requirements for a tight-fitting joint would make the precast units more costly, and some sort of mechanical tie between units would also be necessary—either weld plates or tie rods. The only apparent advantage is that this construction could be accomplished in cold weather.

Diaphragms

Typical diaphragms that have been used on integral deck bridges include: 1) field cast concrete; 2) precast concrete attached in the field; 3) precast concrete cast monolithically with the member in the plant; and 4) steel bracing.

Previous studies have indicated that the value of diaphragms in concrete bridges is questionable. Many of the bridges investigated had no diaphragms between supports and some (Fig. 24) even eliminated them at the support.

Significant cost savings can be realized by eliminating diaphragms and further study on this matter is encouraged. This controversy is beyond the scope of this research project.

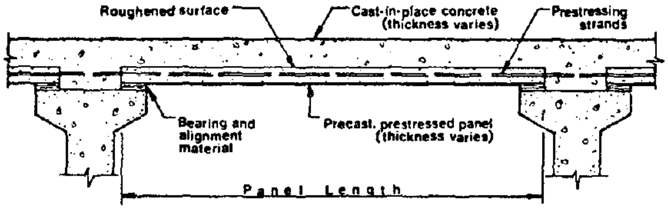


Fig. 1 Typical construction using stay-in-place forms.

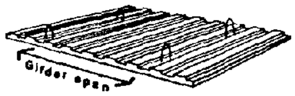


Fig. 2 Typical ribbed panel (dimensions vary).

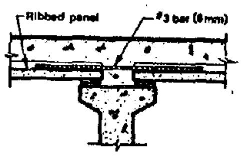


Fig. 3 Construction using continuity bars.

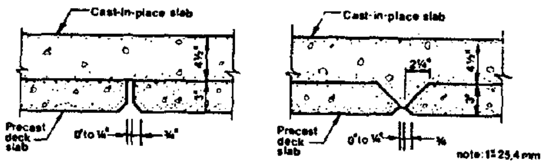


Fig. 4 Joint details for concrete deck panels—Pennsylvania State University tests.

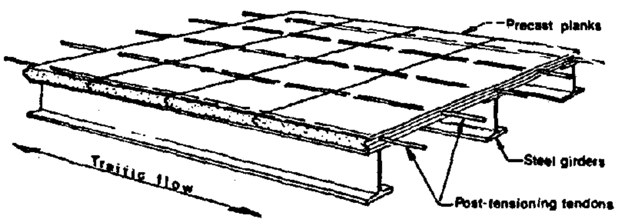


Fig. 5 Schematic of Purdue test bridges.

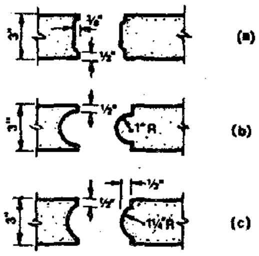


Fig. 6 Joint types investigated for Purdue tests.

Fig. 7 Experimental bridge at Knightstown, Indiana.

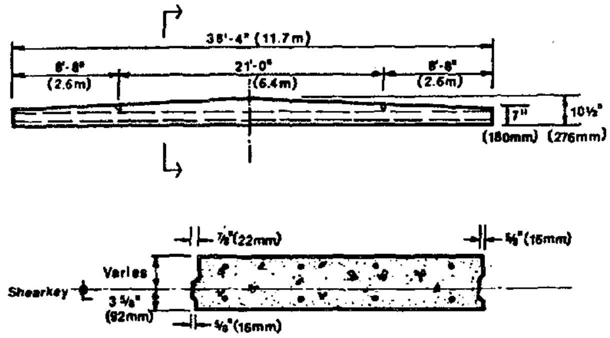


Fig. 8 Elevation and shear key details—slabs used on Knightstown bridge.

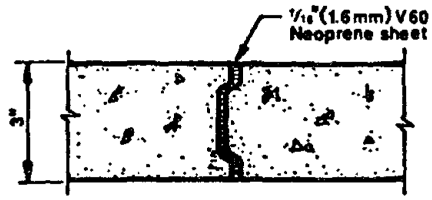


Fig. 9 Joint detail—Indiana tests.

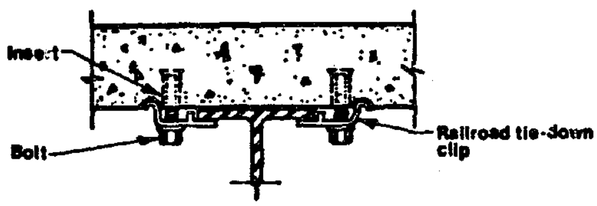


Fig. 10 Tie-down connections used on Indiana Highway Commission experimental bridges.

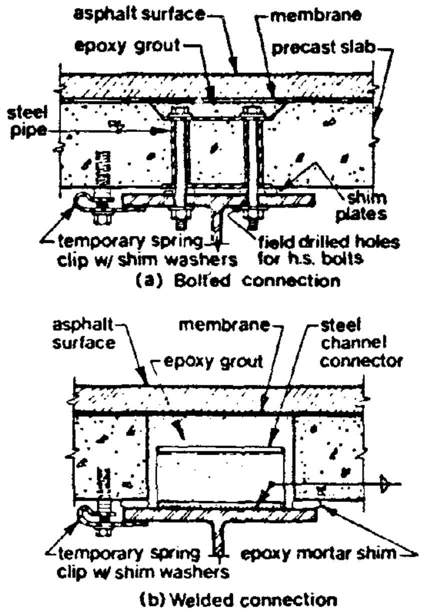
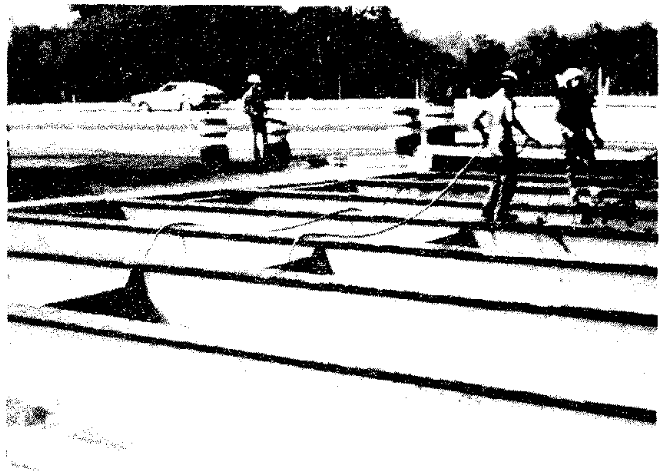
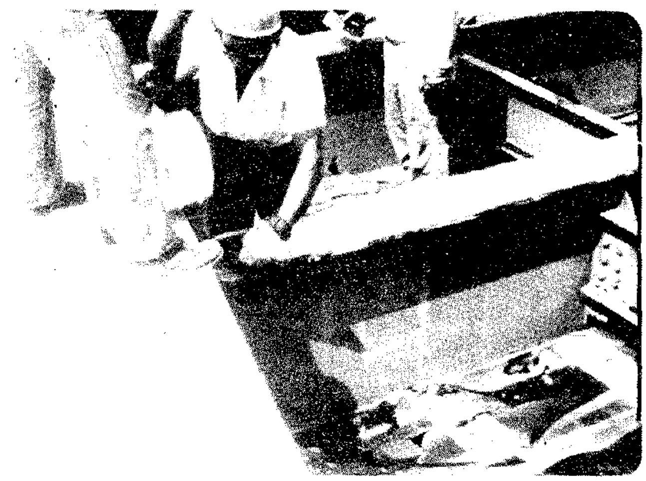


Fig. 11 Connections used on New York Thruway experimental bridge at Amsterday, New York.



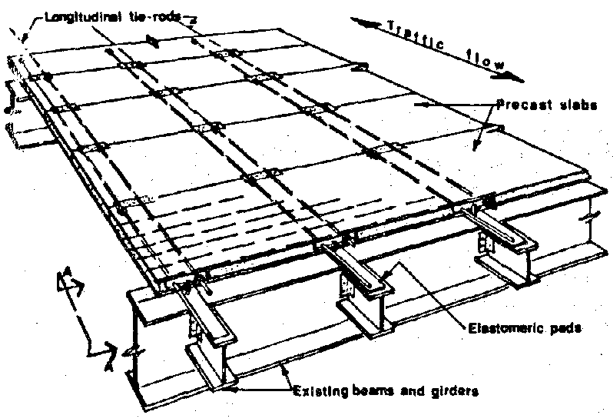
a. Flange preparation.



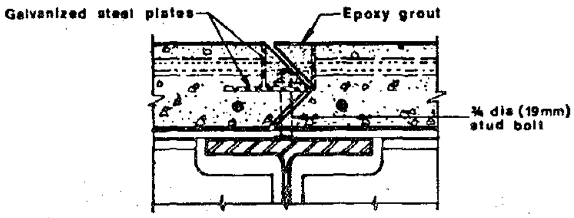
b. Placing epoxy mortar bed.

c. Slab placement.

Fig. 12 Construction of Krum Kill bridge New York Thruway.

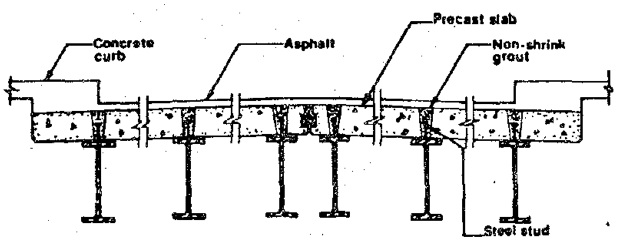


(a) Schematic showing construction

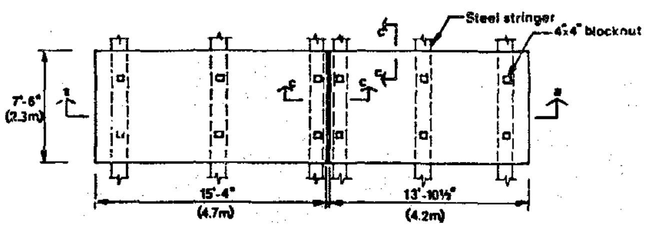


(b) Section A-A

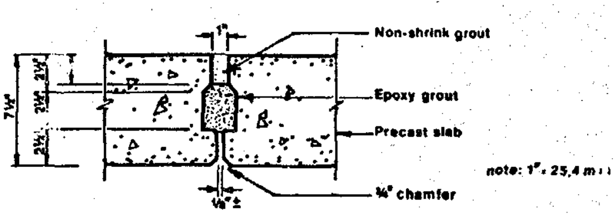
Fig. 13 Rondout Creek bridge deck, Kingston, New York.



(a) Transverse section



(b) Plan



(c) Shear key detail

Fig. 14 Delaware River bridge, New York—Pennsylvania.

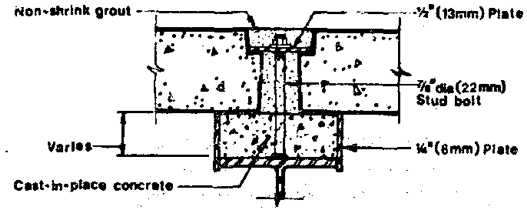


Fig. 15 Connection detail, Erie County, New York bridge.

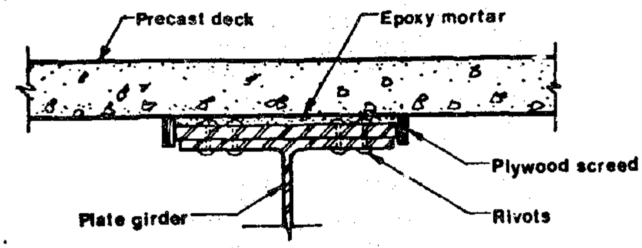
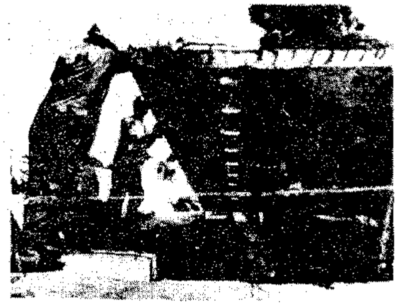


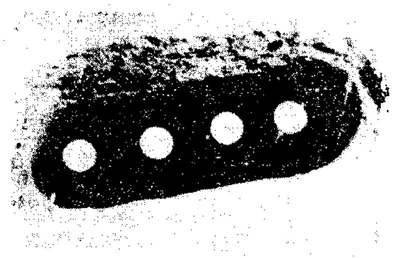
Fig. 16 Connection detail, Santa Fee Railroad bridges.

(a) Old deck removed



(b) New precast deck section placed

(c) Studs welded through blockouts in slab



(d) Stud connection before grouting

(e) Grouting blockouts

Fig. 17 Deck replacement, Interstate 80 overpass, Oakland, California
Courtesy Nelson Div., TRW Co.

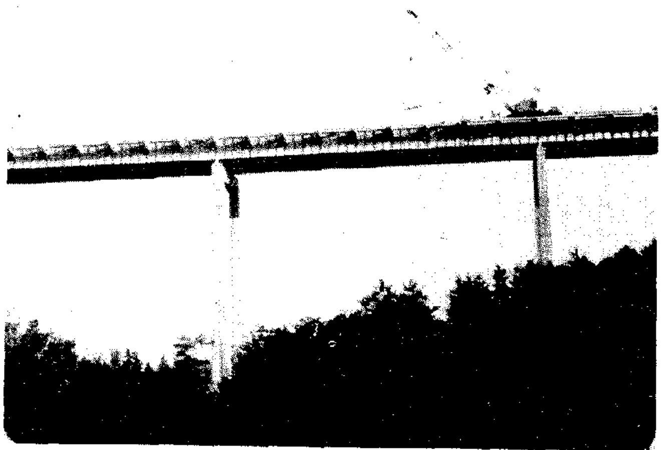


Fig. 18 Clark's Summit bridge, Pennsylvania Turnpike.

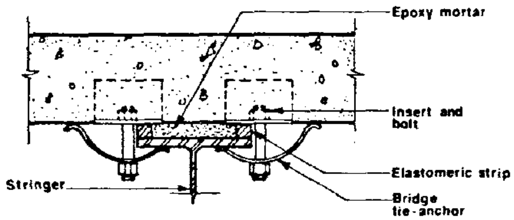


Fig. 19 Connection of slab to stringer, Clark's Summit bridge.

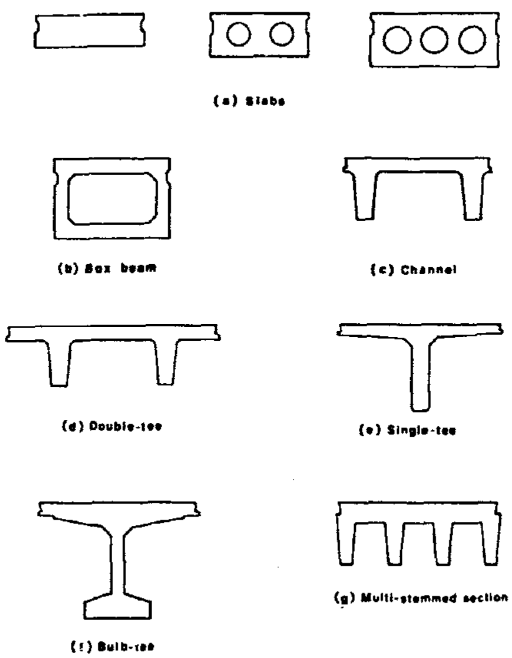


Fig. 20 Sections used for integral deck bridges.

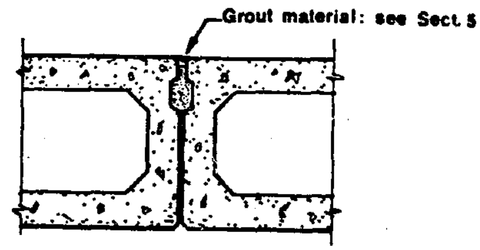


Fig. 21 Longitudinal grouted shear key

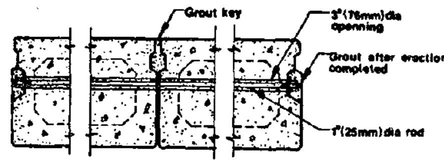
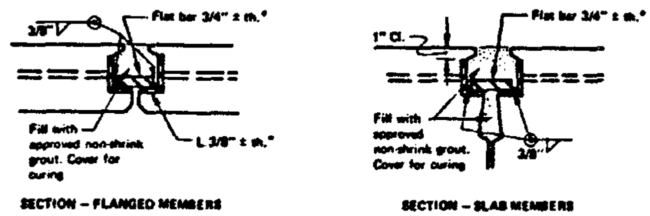


Fig. 22 Transverse tie rod.

Fig. 23 Recessed welded connections.



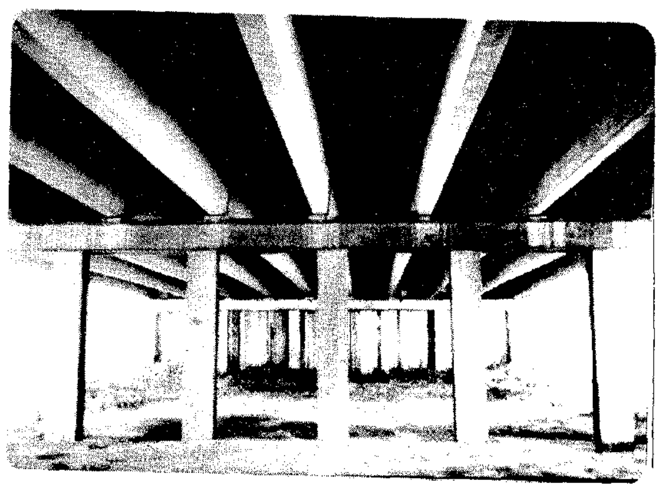


Fig. 24 Underside of double tee bridge with no diaphragms.

POLYMER CONCRETE AND ITS APPLICATION IN BRIDGE MAINTENANCE

Dr. L. Srinivasa Iyer

Associate Professor, Dept. of Civil Engineering,
South Dakota School of Mines and Technology,
Rapid City, South Dakota, U. S. A.

Summary

Polymer Concrete is a monolithic material composed of aggregates bound together with a polymeric compound. Its chief advantage lies in its quick setting and fast development of strength without any special curing, thereby reducing significantly, traffic closure time on busy highways and airport runways.

The paper discusses the development of an optimized polymer mix at South Dakota School of Mines & Technology and its properties compared with the commercial mixes available in U.S.A. Simulated patch work in the laboratory are evaluated for strength properties, finishability, workability and thermal compatibility with Portland Cement Concrete. Field application of polymer concrete to maintenance of pavements, runways and bridge decks are in progress.

INTRODUCTION

During the past decade, considerable research has been done throughout the United States and other countries in the development of Polymer Concrete. Polymer concrete is a monolithic, composite material formed by mixing aggregates and monomers and subsequently polymerizing the monomers to form a strong and durable material. The monomer polymerizes and acts as the binder. The main advantage of Polymer Concrete over Portland Cement Concrete is that Polymer Concrete develops very high strength without curing in less than two hours as compared to the 28 day strength of Portland Cement Concrete.
This property of Polymer Concrete enhances the potentiality of the material in various construction applications. Studies have shown that polymer concrete exhibits very high compressive, tensile and flexural strengths and resistance to most of the chemicals, water absorption and freeze-thaw deterioration.[4]

One of the major problems faced by the Highway Systems of the United States and other countries is the deterioration of reinforced concrete bridge decks due to the penetration of deicing salts and subsequent corrosion of reinforcing steel. This calls for the development of a durable overlay material that is resistant to water absorption and freeze-thaw deterioration. Also the situation warrants the use of a material which involves least work time from start of repair to resumption of traffic, as the cost of detouring traffic is very high. The same requirements apply in the case of airport runway maintenance too. Polymer concrete with its exceptional properties has a great potential to qualify to meet the above requirements.
This paper deals with the development of polymer concrete mix at South Dakota School of Mines and Technology, its properties and its application to the bridge deck maintenance.

POLYMERIZATION

A polymer is a large molecule built up by the repetition of small simple chemical units called monomers.[2] The monomer which is in liquid form is solidified on polymerization to a solid state. There are different methods[1] of polymerization like thermal-catalytic, promoter-catalytic and polymerization by radiation. In the study described in this paper, promoter-catalytic method of polymerization was adopted as it does not need any special equipment for external heating or radiation.

Chemicals

The chemicals required for the production of polymer concrete are hazardous and sufficient precautions should be taken during handling and storage. The basic chemicals involved are monomers, initiators and promoters[1].

Monomers

Monomers are small simple chemical units whicy by repetition build up large molecules called polymers. Some of the widely adopted monomers for the production of Polymer Concrete are Methylmethacrylate (MMA), Butyl Acrylate (BA), Styrene, Polyester styrene and Trimethylolpropane trimethacrylate TMPTMA. TMPTMA acts as a cross-linking agent and is primarily used to increase the rate of polymerization.

Initiators

Initiators behave as catalysts for the formation of polymer chains. These are usually organic peroxides. The most commonly used initiator is Benzoyl Peroxide (BPO). This is available in the form of granular solids or plastic emulsion or powder.

Promoters

Promoters are chemicals used to accelerate the polymerization process. Promoter increases the decomposition rate of the initiators and permits polymerization to occur at ambient temperatures without the addition of external heat. Most commonly used promoters with MMA monomer system are Dimethyl aniline (DMA) and/or Dimethyl-p-tolidine (DMT).

DEVELOPMENT OF POLYMER CONCRETE

There are two methods of making polymer concrete (using the monomer-initiator-promoter system of polymerization) known as prepacked method and premixed method. In the prepacked method the monomer, initiator-promoter system is poured in through pre-packed aggregate matrix till saturation. The drawback of this method is that, when the monomer system is poured, the finer aggregates tend to get deposited at the bottom of the mold thus resulting in segragation. This leaves the top surface rough and uneven. An improvement over this method is that the monomer system is premixed with the aggregate matrix and yields good finish but results in loss of monomer due to evaporation. This technique is modified by changing the chemical constituents so that a thin film is formed on the surface, thus minimizing evaporation loss. Three companies, namely, E.I. Dupont De Nemours & Company, Adhesive Engineering Company and Transpo-Materials Inc. are making the new Polymer concrete as described above.

In the design of polymer concrete mix the aggregates should be so proportioned as to have minimum voids. This will require minimum monomer system and will result in minimum shrinkage and economy. A special gradation of fine aggregates is used in this study to optimize the mix on the basis of minimum monomer loading and good workability. The gradation of the fine aggregate is shown in Fig. I. This specially graded fine aggregate is mixed with the promoter (powder form), fine marble powder and coloring agent to form a granular mix to which monomer system is added to produce polymer mortar. This granular mix is extended by adding regular coarse aggregate satisfying ASTM C33-76 to form polymer concrete. This method is less cumbersome and also, the dry materials can be premixed and stored for ready use.

LABORATORY TESTS

Three commercial mixes and the mix developed at South Dakota School of Mines and Technology (SDSM&T) were tested for exothermic temperature, tensile, compressive and flexural strength. Generally the procedure used for testing was as per applicable ASTM methods for testing the strength of Portland Cement Concrete. The total curing shrinkage of these mixes was determined by a proprietary method developed by the author. The typical time versus exothermic temperature plot is shown in Fig. II. The three hour and 24 hour strength test results are shown in Table I-A and I-B for various mixes.

Fig. I. SPECIAL GRADATION FOR FINE AGGREGATES

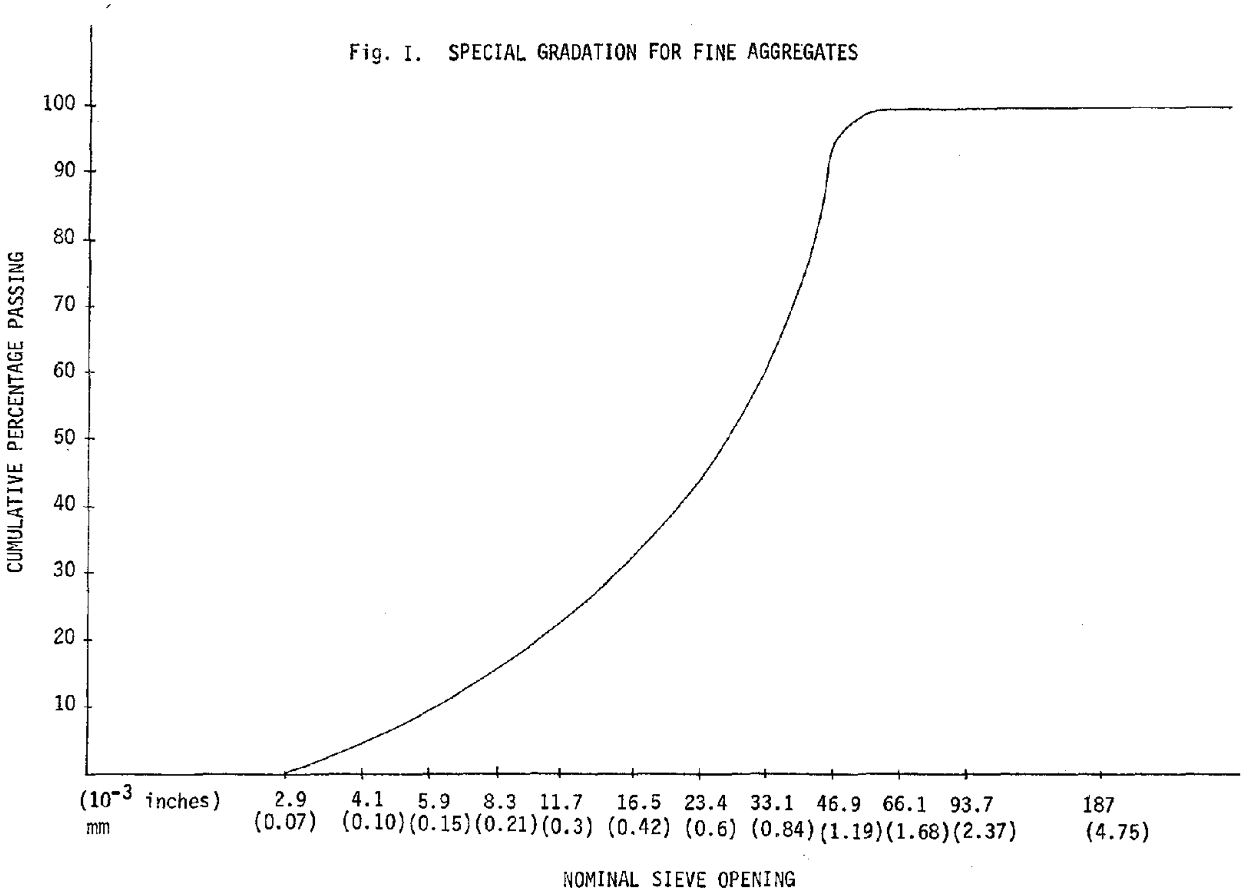


TABLE I-A

MECHANICAL PROPERTIES OF POLYMER MORTAR

MIX PRODUCT	TYPE OF TEST	PEAK Time (Min)	PEAK Temp °F (°C)	AGE OF SPECIMEN IN HOURS	AVERAGE STRESS Psi	AVERAGE STRESS MPa
DuPont	Comp.	50	95 (35)	3 24	4750 6603	32.75 45.53
	Tens.	44	93 (34)	3 24	705 820	4.86 5.65
	Flex.	38	92 (33)	3	2965	20.44
Transpo Materials	Comp.	27	116 (47)	3 24	6045 6180	41.68 42.61
	Tens.	25	97 (36)	3 24	470 580	3.24 3.99
	Flex.	23	103 (39)	3	285	1.97
Adhesive Engg.	Comp.	26	102 (39)	3 24	5460 5400	37.65 37.23
	Tens.	26	93 (34)	3 24	835 935	5.75 6.45
	Flex.	27	105 (41)	3 24	2440 1320	16.82 9.10
School of Mines	Comp.	44	98 (37)	3 24	8435 9635	58.16 66.43
	Tens.	44	82 (28)	3 24	1115 1415	7.69 9.76
	Flex.	45	88 (31)	3 24	2795 2910	19.27 20.06

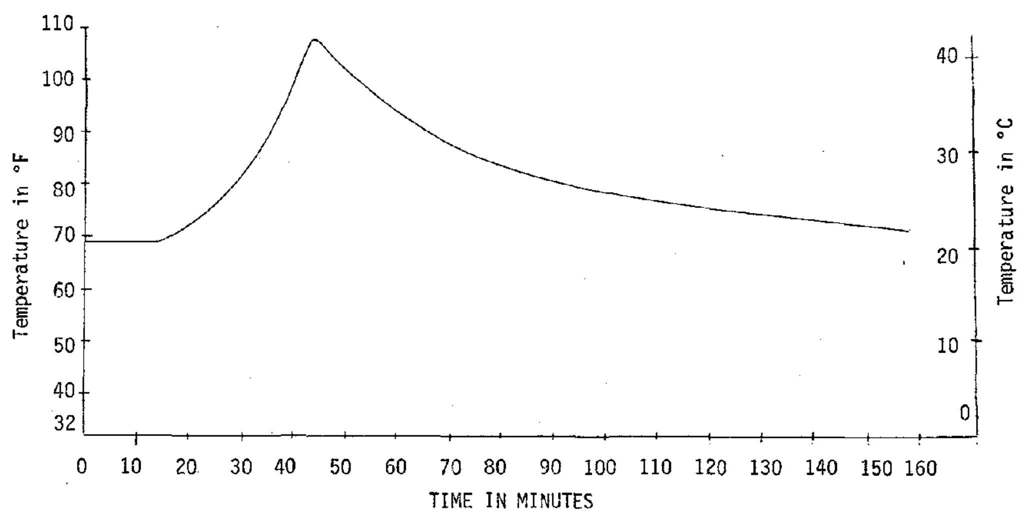


FIG. II TYPICAL TIME VERSUS EXOTHERMIC TEMPERATURE PLOT

TABLE I-B

MECHANICAL PROPERTIES OF POLYMER CONCRETE

MIX PRODUCT	TYPE OF TEST	PEAK Time (Min)	PEAK Temp °F (°C)	AGE OF SPECIMEN IN HOURS	AVERAGE STRESS Psi	AVERAGE STRESS MPa
DuPont	Comp.	58	95 (35)	3 24	5820 5675	40.13 39.13
	Tens.	58	95 (35)	3 24	970 1045	6.68 7.20
	Flex.	58 52	91 (33) 110 (43)	3 24	1620 1545	11.17 10.65
Transpo Materials	Comp.	25	121 (49)	3 24	5710 5995	39.37 41.34
	Tens.	25	121 (49)	3 24	1035 1055	7.14 7.27
	Flex.	25	121 (49)	3 24	1385 1710	9.55 11.79
Adhesive Engg.	Comp.	25	111 (44)	3 24	5325 5865	36.72 40.39
	Tens.	25	111 (44)	3 24	880 890	6.07 6.14
School of Mines	Comp.	56	92 (33)	3 24	6650 7320	45.80 50.47
	Tens.	56	92 (33)	3 24	1560 1650	10.75 11.37
	Flex.	59	106 (41)	24	2500	17.24

These mixes were developed for a temperature range of 40°F to 110°F (4°C to 43°C). A mix for lower temperature ranges of 0°F to 40°F (-18°C to 4°C) is being developed and tests are in progress. The shrinkage values of the four polymer mortar mixes are shown in Table II.

Simulated patch work using polymer mortar and concrete was studied in the laboratory for bond, finishability and workability of the four mixes. A tack coat that will have good bond between the original concrete and polymer patch material was developed using the same monomer system and coupling agents. This tack coat was applied on the preformed potholes in the precast slab prior to the placement of polymer mortar or concrete as shown in Fig. III. Small quantities of Polymer concrete can be mixed in a wheel barrow where as larger quantities can be mixed in a regular concrete mixer as shown in Fig. IV. The finishing of the surface is shown in Fig.V. After 24 hours the slabs were cored and the patch material over base concrete were tested for bond. All the cores had failed in tension at base concrete indicating the bond between base concrete and polymer concrete was very good. Two slabs from two mixes (SDSM&T and Transpo) with patches were subjected to five freeze-thaw cycles as per ASTM C884-78 (modified) for thermal compatibility. After the completion of five cycles boundary cracks were noticed in the case of patches made with Transpo Company where as no crack or destress was noticied in the slab using SDSM&T mix. The patch size was 18 to 24 in. (45 to 60 cms).

FIELD TESTS - PILOT PROJECTS

On the basis of laboratory studies, the SDSM&T polymer mix was selected for field applications. Three locations in the state of South Dakota were used for testing the polymer concrete patch work in the field as part of the pilot project. One location was on the runway and taxiway of airport, where several pot holes were repaired using SDSM&T polymer concrete mix with one inch maximum size coarse aggregate. The monomer loading was only five percent of the total mix. Total down time was limited to four hours and the traffic was resumed one hour after the placement of polymer concrete. These patches were done at air temperature of 70 to 90°F (21°C to 32°C).

The other two locations were on bridges, one within the city limits on a busy street and another one on an Interstate highway system near a city where traffic was very high. Total down time in each case was less than three hours and the work was completed in the relatively non-busy time of the day (between 9 A.M. and 12 noon).

TABLE II

TOTAL SHRINKAGE RESULTS OF POLYMER MORTAR

SPECIMEN SIZE: 1½" x 1½" x 11¼" (38 x 38 x 286 mm)

MIX PRODUCT	MONOMER PERCENT BY WEIGHT	EXOTHERMIC PEAK °F (°C)	PEAK TIME (Mins)	AMBIENT TEMP °F (°C)	SHRINKAGE mm/mm $\times 10^{-4}$
Transpo Materials -17-	8	109 (42.7)	37	70 (21.1)	6.75
Adhevise Engineering -20-	8	108 (42.2)	36	70 (21.1)	5.40
School of Mines	9.4	108 (42.2)	45	70 (21.1)	3.93


Fig. III The tack coat being applied to the pothole.


Fig. IV Mixing of the polymer concrete.


Fig. V Leveling of the mix after placing.

All the patches in three locations were evaluated for strength of the mix, cracks, delaminations and bond strength on core samples. No delamination was noticed on any of these patches even after one year of usage. Some minor cracks were noticed when the size of the patches was greater than 3 to 4 feet (90 to 120 centimeters). These cracks were repaired using the tack coat and they did not reappear after one year. The evaluation of these patches and more patch works on the runway of the airport are in progress.

CONCLUSIONS AND RECOMMENDATIONS

Polymer concrete using the monomer - initiator - promoter system proved to be an efficient system except for the low flash point of the monomer which is near 52°F (11°C). The total curing shrinkage in combination with the differential temperature stresses limit the size of the patches where extreme temperature changes occur. The polymer concrete mix developed at SDSM&T seems to be equal or better than the three commercial mixes studied in this project. Further studies are in progress with new monomers with high flash point and lower shrinkage to offset the disadvantage of cracking and flammability.

ACKNOWLEDGEMENT

This study was funded by South Dakota Department of Transportation and Federal Highway Administration, U.S.A. Thanks are due to E.I. Dupont De Nemours & Company, Transpo Materials Inc. and Adhesive Engineering for their supply of chemicals.

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INNOVATIVE METHODS OF UPGRADING STRUCTURALLY AND GEOMETRICALLY DEFICIENT THROUGH TRUSS BRIDGES

Gajanan M. Sabnis
Professor of Civil Engineering,
Howard University, Washington, D.C.

Lloyd Cayes
Structural Engineer,
Federal Highway Administration,
Department of Transportation,
Washington, D. C.

Hemant Modi
Structural Engineer, Sheladia Associates, Inc., Riverdale, Maryland.

SUMMARY

This paper deals with the rehabilitation of the Through Truss bridges which have been constructed decades ago. Many of these bridges are too narrow and structurally limited to accomodate the present traffic. The main problem of these older bridges is that they are substandard (i.e. deficient), especially for the increased size of vehicles and traffic volume. The work presented here is an outcome of the Federal Highway Administration (FHWA) research project, concerned with the development of possible solutions to upgrade the Through Truss bridges. Several methods were developed to solve the problem in a cost effective manner and are presented along with examples. The general idea of upgrading follows from (a) Use of composite structure to restore and increase original strength; (b) Reduction of roadway dead load by using cables that are attached between the truss members and the ends of truss support; and (e) Conversion of a single span truss into a two or three span continuous truss.

INTRODUCTION

Bridges form an integral part of modern road networks. Many of these bridges are too narrow and structurally inadequate to accomodate the present traffic. Considerable attention has been given to the safety of the public. Current Federal Regulations require periodic inspections and rating of each bridge. Following these, the federally funded bridges have received the attention necessary to maintain safety and serviceability. The main problem of older bridges, and in particular the truss bridges, stems from the growing numbers which are aged and substandard (i.e., deficient) combined with the increased vehicle size and traffic volume. The situation calls for an evaluation of the whole spectrum of "truss bridge engineering", in terms of (a) Inspection and assessment of bridges for present usage; (b) Evaluation of their load-capacity and maintenance; and (c) Determination of their level of safety based on structural and/or geometrical adequacy. Until recently, little attention had been given to the problems associated with the maintenance and rehabilitation of older structures on the highways. However, the rehabilitation of bridges which are deficient from either a structural or a functional point of view is an essential alternative if cost-effective compared to the total replacement.

The overall goal of this research investigation was to develop methods to upgrade the capacity of a structurally and geometrically deficient (functionally obsolete) Through Truss bridge. It also addresses to the need for feasible and economical rehabilitation and replacement procedures so that these inadequate structures can be made safe and fully serviceable.

In order to achieve the above overall objectives of this research, the work (25) was carried out as follows:

(a) Review of current practices on extending the service life of existing bridges;

(b) Identify common structural and functional (geometrical) deficiencies;

(c) Develop methods to upgrade the capacity of existing Through Truss bridges;

(d) Prepare construction drawings and specifications; and

(e) Cost comparison of rehabilitation and replacement.

In this paper, only (a), (b), and (c) are presented.

REVIEW OF CURRENT PRACTICES

A number of investigators have worked on the rehabilitation works and practices followed in several states for bridges. These include, Berger (1978), NCHRP Report 12-20 (1980) and Newlon (1978), which covered considerable information on the practices and examples as well as the historical values of some of the structures for their rehabilitation.

The OECD Research Report (1979) provides information on the evaluation of load carrying capacity mainly in European practice, while inspection manuals (NY 1978), (MD 1978), and (WV 1978) and others, provide some insight into the maintenance programs in some of the states.

The NCHRP Report 12-20 (1980) has identified some of the deficiencies for truss structures along with possible solutions and their limitations. These methods in this report identify the solutions on an element by element basis rather than focusing on the problem of upgrading the bridge as a whole.

Two projects (1980a, 1980b) in the State of West Virginia, involving the inspection and development of recommendations for maintenance of truss bridge serviceability were carried out by the author's office. As in the NCHRP Report, these projects also addressed the structural deficiencies caused by age and other factors found in the various elements of the trusses.

NCHRP Digest 98 (1977) addressed the issue of safety at narrow bridge sites and developed a bridge safety index (BSI) for determining priorities in dealing with problems of bridges with restricted widths. The index was developed by combining the various factors affecting geometrical deficiency and assigning them suitable values so that the index would represent a BSI. The recommended corrective measures were based primarily on engineering judgment and the practice locally in Texas.

Newlon's (1978) work mainly focusses on the rehabilitation of (truss) bridges from the point of view of their historical significance. He developed a nummerical rating system in three categories: documentation (age and builder), technology, and environmental factors. Ideas on the rehabilitation of some old bridges, contained in this report, were applicable to the present project.

Bridge maintenance manuals by AASHTO (1978), the States of West Virginia (1978), Maryland (1978), and New York (1979) essentially deal with similar procedures that should be followed for inspection and maintenance of bridges.

The Delaware Department of Transportation categorizes the reasons for rehabilitation in the form of priorities as follows:

Priority 1 - Rehabilitation that will insure safety and integrity of the structure.

Priority 2 - Renovation recommended to reduce the probability of future deterioration and maintenance.

Priority 3 - Repairs of minor items to improve appearance and functioning of the structure, and routine maintenance procedures.

The Indiana State Highway Commission and the Louisiana Department of Transportation emphasize that collisons are usually more serious than corrosion for they damage new as well as old structures, whereas corrosion damage usually involves old structures and those made up of small members or thin sections like lightweight truss members of small stringers.

The Ohio Department of Transportation upgrades the bridge to the AASHTO HS-20-44 standard loading capacities for which a repair project is proposed.

The Department of Transportation of Illinois states that the rehabilitation procedures should consider many factors: urgency, costs, material accessibility, density and the type of traffic, age and physical condition of base material, size of project, and whether or not the works are to be accomplished by in-house staff or contractors.

The AASHTO Specifications for rating and determining the capacity of existing bridges is a "weak-link specification". They allow a 125% overstress for existing members in good condition based on computed live load stresses. This type of rating often tends to be overly conservative and should allow for the inclusion of reserves of strength in the form of testing to determine the actual live load stress conditions. This requires a thorough inspection and should be allowed only after the repair of any deficiencies.

DEFICIENCIES IN THROUGH TRUSS BRIDGES

The second phase of the research strictly involved developing methods of upgrading structurally and geometrically deficient Through Truss bridges. Bridge deficiencies evolve from a variety of situations and conditions. There are also other factors such as intensity of traffic, environmental conditions and basic design assumptions which affects the integrity of structure to a great extent. However, the maintenance of bridge plays an important role in keeping up the standard of bridge.

Before the actual methods are presented, it is essential to recognize the deficiencies in through truss bridges. The deficiencies include (a) structural, (b) geometric, and (c) those related to safety.

(a) A bridge is designed to carry certain standard loads. Over the period of the last few decades, loads have increased, which affect adversely the ability to carry these loads and makes it structurally deficient.

Structural deficiencies occur in a variety of situations and conditions. In particular, for truss bridges, it becomes an involved problem, since they are built with a large number of members. Some of them may be over-designed and often, may get overstressed at critical locations, when subjected to the load heavier than they were designed for.

(b) Geometric Deficiencies

Many of the existing steel bridges in the United States were built before the adoption of modern design standards, mainly based on the width. In many instances the bridges were left untouched due to the expenses involved in increasing width; this resulted in geometric deficiency, due to several reasons including narrowness of such bridges, when new four lane bridges were required.

Geometrically deficient bridges also often have a clearance problem or an inadequate approach way; such bridges can no longer serve the public safely.

(c) Safety Related Deficiencies

These are the deficiencies which would jeopardize the safety of the vehicle as it passes over the bridge. Many of these are inherent geometric deficiencies that were built in as a result of the initial design (roadway width, clearance, alignment, etc.). Other deficiencies are such as bridge railing, approach guard rail protection, traffic control devices, quality of bridge deck (wearing surface breakdown), approach slab settlement, etc.

A number of stop-gap measures have been attempted to hazardous bridge sites besides widening of the roadway. These measures are briefly pointed out as follows:

(a) Replacement of bridge rail;

(b) Installing the approach guard rail;

(c) Installation of narrow bridge and speed limit signs;

(d) Re-routing of heavier vehicles;

(e) Realignment of roadway; and

(f) Placing edge lines and transition markers.

These recommended corrective measures are based primarily on engineering judgment.

(a) Stiffening of Compression Member (Structural)

In this method the main idea is to reduce the length of member by using sub panel at certain locations. This reduces the slenderness ratio and results in the increase in allowable stress in the member. By reducing the length of compression member to half, we are able to increase capacity from HS-15 to HS-20 loading.

This was achieved in a warren truss as described in Fig. 1.

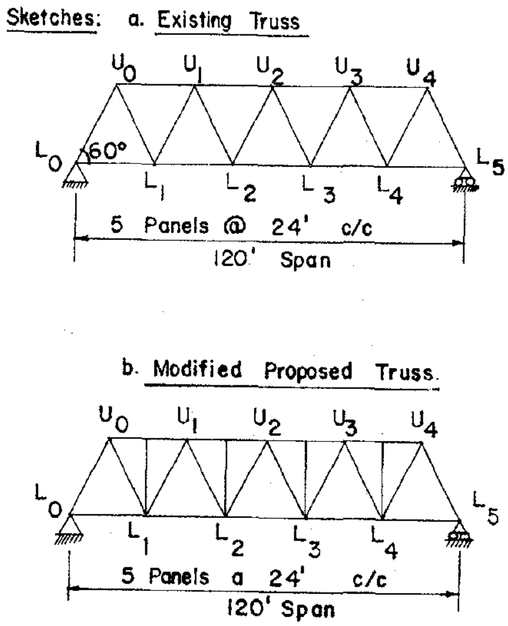


Fig. 1

(b) Method of Prestressing (Structural)

Basically the idea is to induce stresses of opposite nature compared to those resulting from applied live loads. Prestressing in truss is accomplished by applying a compressive force to the bottom chord which, under normal loading, would be in tension. The truss is thus prestressed so that the induced axial forces are the reverse of those produced by normal loads. In this method, the cables are strung along the truss tension member and attached to the end of the member or to the connecting pin. Turnbuckles are used to induce tensioning in cables. The compression thus introduced in bottom chords permits the member to carry additional live load (Fig. 2).

The analysis is based on applying a unit tension to the cables attached to the vertical strut which eventually props the bottom chord.

Method of Prestressing

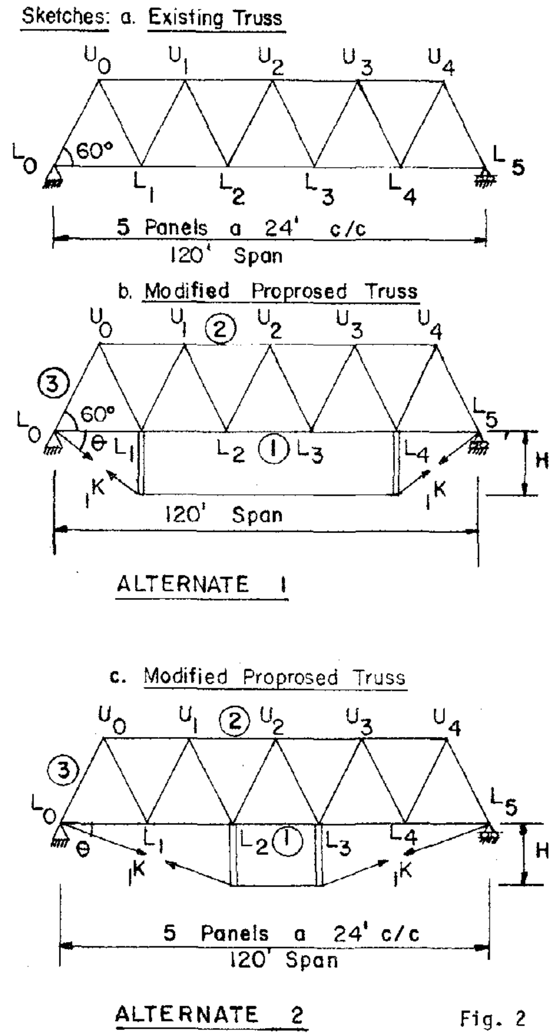


Fig. 2

Various values of forces in critical members due to unit tension in the cable are presented Table 1.

(c) Shifting of Support (Structural)

At times when the support is eroded or damaged, the truss may become unsafe to carry the traffic, which makes it necessary to develop a support to the truss at an alternate location.

This method of shifting the support either at only one end or at both the ends, reduces the effective span and thus the forces in the truss members. Analysis is performed for the truss with two ends as cantilever. By reducing the span, forces on the average truss members can be reduced by 40% to 50%.

	H Feet	Deg.	Tension Kips	Hor. Force Kips	Vert. Vorce Kips	Force in Critical Member 1	2	3
ALTERNATE 1	3.0	7.125	1	0.992	0.124	1.135	0.142	0.142
	6.0	14.03	1	0.97	0.242	1.24	0.278	0.278
	12.0	26.56	1	0.894	0.447	1.408	0.514	0.514
	18.0	36.86	1	0.799	0.6	1.49	0.69	0.69
ALTERNATE 2	3.0	3.57	1	0.998	0.0623	1.14	0.143	0.071
	6.0	7.125	1	0.992	0.124	1.277	0.285	0.142
	12.0	14.03	1	0.97	0.242	1.53	0.556	0.278
	18.0	20.55	1	0.936	0.351	1.74	0.807	0.403

Table 1

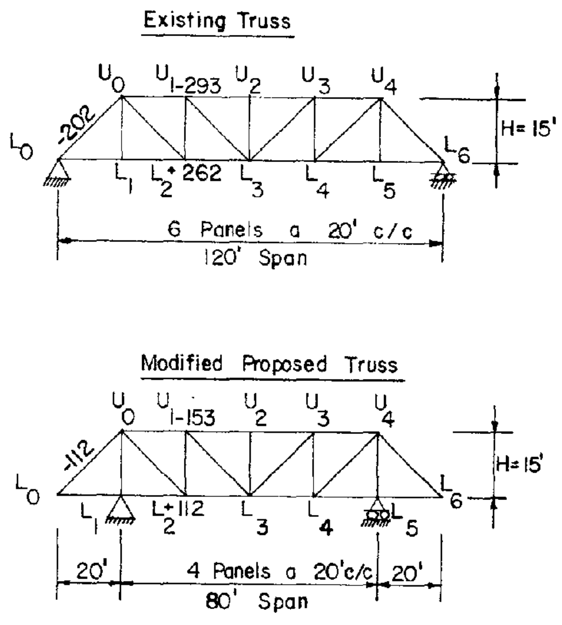


Fig. 3

(d) Addition of Central Support (Structural)

This method mainly reduces the span and makes the truss continuous. By doing so, forces in the members are reduced substantially. The method is particularly suitable when placing such central support, or at any other intermediate joint would be relatively easy.

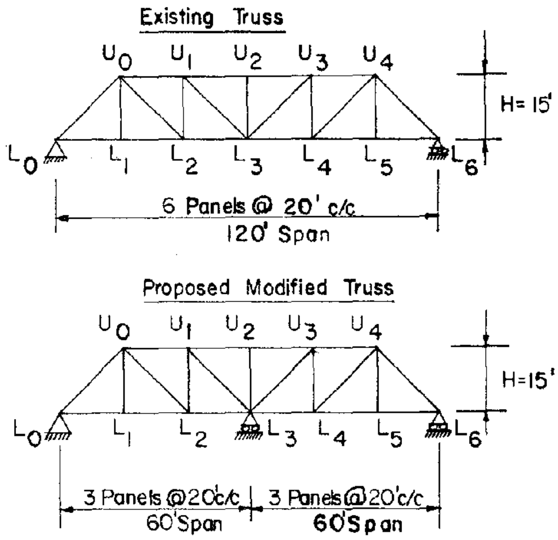
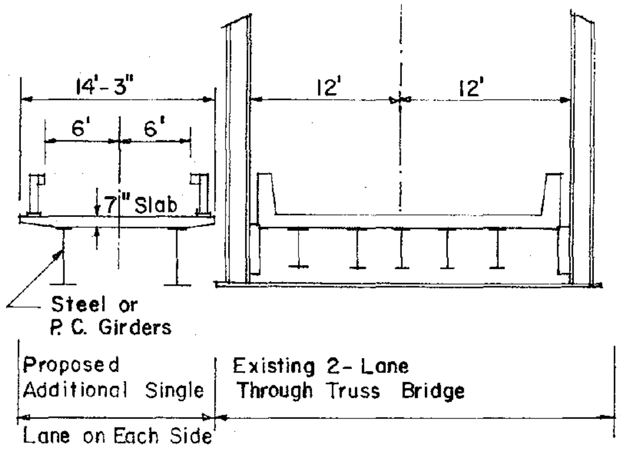


Fig. 4

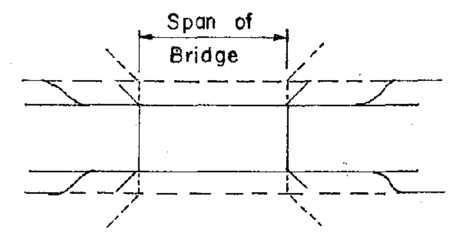
(e) Additional Lanes for Heavy (Truck) Loading (Geometric)

Many of the existing steel bridges in the United States were built before the adoption of modern design standards. To accommodate them, the roadways were widened; however, in many instances the bridges were left untouched due to the expenses involved in increasing width; geometric deficiency thus stemmed out of the narrowness of such bridges, intensity of traffic and existence of four-lane highways.

Another method of rehabilitation that was considered is an addition of a single lane on each side of the existing bridge in order to carry the heavier HS-20 truck loads, but without transferring the load to main bridge spans. The existing bridge is then to be used for lighter vehicles only. This will result in a four-lane bridge from the original two-lanes. We must recognize that the additional bridge lanes are kept independent of the original bridge without stressing it beyond it's present loading.



TYPICAL CROSS SECTION



KEY PLAN

———— Existing Bridge

- - - - - - - Proposed New Bridge

Fig. 5

(f) Increase Vertical Clearance

At certain locations on old steel bridges, vertical clearance causes a big problem due to restriction on the recent heavy commercial traffic, and therefore a geometric deficiency in the bridge.

In this method, extra vertical clearance is obtained by rearranging the inclined bracings. The alternate arrangements are, (1) provide horizontal member to take the same thrust, and (2) provide horizontal member at the bottom at a distance of 3'-0" above the original location, thus giving an additional three feet clearance. The method is shown in detail in Figure 6. It must be noted that between Figure 7 and Figure 8 is that in order to achieve the latter, one has to convert a truss into a frame and analyze accordingly.

Increase Vertical Clearance

Existing Truss

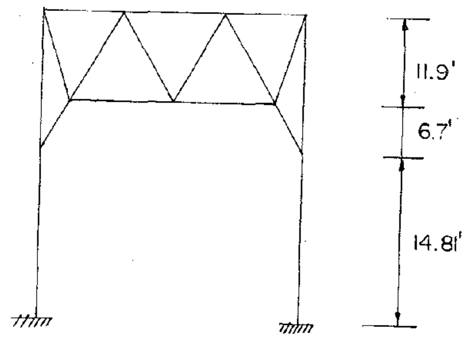


Fig. 6

Modified Proprosed Truss

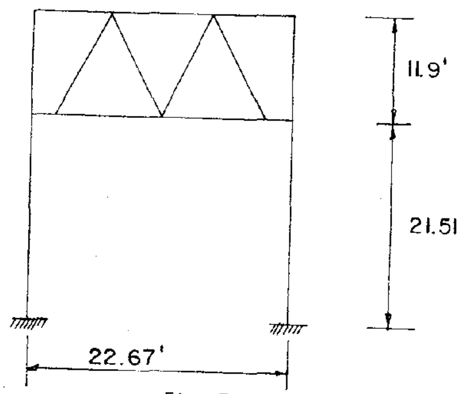
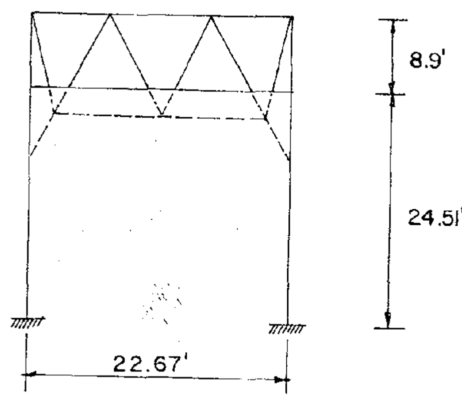


Fig. 7

Modified Proprosed Truss



Note: - - - - - - Portion to be Removed

Fig. 8

EXAMPLES

Stiffening of Compression Member

1. Design of compression member mainly based on its unbraced length (ratio of $K\ell/r$), thus giving certain value of allowable stress for that member. In this method by subpaneling, the length of member is reduced which increases the allowable stress in that member. In general, if the length of compression number is reduced to half, its capacity was found to increase from HS-15 to HS-20 loading without affecting the member, e.g.,

   Member U_1U_2 HS-15 force - 219^k (Fig. 1)

   Member Double L^s 8" X 8" X 3/4"

   r_{min} = 2.47" A = 22.9 in^2

   $K\ell$ = 288"

   $$\frac{K\ell}{r_{min}} = \frac{288}{2.47} = 116.6 \therefore F_{all} = 10.77 \text{ ksi}$$

   $$f_{act} = \frac{21.9}{22.9} = 9.56 \text{ ksi} < 10.77 \text{ ksi} \therefore$$

   Thus, by subpaneling the truss, the member, U_1U_2, is upgraded by reducing the length by half.

   Force in U_1U_2 due to HS-20 loading - 292^k

   $$\frac{K\ell}{r_{min}} = \frac{144}{2.47} = 58.3 \therefore F_{all} = 17.59 \text{ ksi}$$

   $$f_{act} = \frac{292}{22.9} = 12.57 \text{ ksi} < 17.59 \text{ ksi} \therefore$$

2. Method of Prestressing

   In the case of trusses, which are subjected to both the dead load (D.L.) and the live load (L.L.), including impact, the proportion of D.L. is reasonably higher than L.L. + impact load. The member selected for prestressing already has stresses due to dead load (before the application of live load). Turnbuckles are used to induce tension in the cables, thus inducing axial forces in the truss members which are the reverse of those produced by normal loads.

The following table shows the effect of unit tension on critical members:

	HS-15 Loading			HS-20 Loading		
	1	2	3	1	2	3
D.L.	139	139	93	139	139	93
L.L. + Impact Load	74	80	43	99	106	58

When tension is applied to the cable, it would counteract the dead load stresses and subsequently reduce member force. When unit tension is applied, e.g., to a 6'-0" high strut, the forces in critical members are:

(1) 1.24^k (2) 0.278^k (3) 0.278^k

For member (2), in order to increase the capacity from 80 to 106 (L.L. + Impact), you need to apply tension (See Fig. 2, Alternate 1).

$$= \frac{(106-80)}{0.278} = \frac{26}{0.278} = 93.5^k$$

Compression in bottom chord =

$93.5 \times 1.24 = 116^k$.

Thus one needs 93.5^k tension in the cables in order to elevate the capacity from HS-15 to HS-20 loading.

CONCLUSIONS

This paper only presents the methods which were investigated during the FHWA research project. The project report (25) deals with these methods in detail and is recommended if one wants to get in-depth to the topic. The main conclusion from this project reached was that it is possible to rehabilitate the "old" through truss bridges using practical and simple methods shown here and through truss bridges need not be replaced to achieve the current highway standards at many locations.

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REHABILITATION OF PELHAM PARKWAY BRIDGE

STRUCTURAL EVALUATION-CONCRETE ARCH SPANS

Avanti C. Shroff
Vice President, Iffland Kavanagh Waterbury, P. C

SUMMARY The Pelham parkway Bridge over Eastchester Bay in the Bronx, New York is a seven span, four lane structure constructed in 1907. The seven spans consist of a moveable span in the center and three fixed approach spans on each end. The moveable span is a double-leaf Scherzer Rooling Lift, bascule structure. The fixed spans are concrete spandrel arches with the roadway placed on fill. Overall length of the structure between ends of wingwalls is about 855 feet. The width of the bridge is 52 feet out to out of parapets.

The entire project was divided into four major phases of work nemely: In-depth inspection of the entire structure, Stress analysis and rating of both the basic structure and bascule machinery,Preparation of reconstruction project report to include recommendations and construction cost estimates along with results of inspection and stress analysis and finally, Preparation of contract documents for the rehabilitation of the bridge.

This paper, the second in a series, discusses Stress Analysis, quality and strength evaluation and rating (Phase 2) of the concrete arches. Petrographic analysis, Compressive strength tests, Air contenttests, Freeze-thaw tests and Chloride tests were performed on samples prepared from thirty-six concrete core specimen obtained in the field. The paper discuss how this test data was used to evaluate the quality and compressive strength of the seventy year old concrete; and then reduced to 28 day strength to compare it with the results of stress analysis.

STRESS ANALYSIS

Existing Construction

The roadway over the arch spans is supported directly on compacted earth fill. This fill rests on top of the main concrete arches and is retained by spandrel walls on either side of the bridge. The spandrel walls rise vertically from the top face of the main arches. Vertical Construction joints in the walls allow the arches to deflect as required without resistance from the walls. Live loads are distributed through the fill to the arch members and then to the foundation.

Essentially, fill material including the pavement, verying in thickness from about 1 foot (30 mm)at the crown to about 6.5 feet (2m) at the spring lines is supported on top of the arches. The supporting concrete arches vary in-depth from 2 feet (61mm) at the crown to about 5 feet (1.5m) at supports and encased steel ribs. These ribs, connected by lattice work with transverse bracing, are spaced 3 feet (91mm) on centers. The 2 feet (61mm) thick spandrel walls are unreinforced concrete with crushed brick veneer on the exposed face. Above the roadway the spandrel walls transition into a 3 feet (91mm) high by 1.5 feet (46mm) thick unreinforced concrete parapet.

Typical pictures of the bridge, towers, piers, and an exposed arch rib are included herewith.

Original Design Concept

The original arch design appears to be based upon the Melan System invented by Joseph Melan of Austria in 1892. The system consists of parallel structural steel ribs connected by lattice work. These ribs are designed to carry the dead load of the arch forms and the wet concrete during construction. The ribs also act as arch reinforcement and support the loads from roadway fill, spandrel walls, pavement and the design live loads.

This structure was designed for a uniform live load of 125 psf (625 kg/sq. m). Vertical joints were provided in the spandrel walls to permit unrestrained behavior within the arch structure.

Basis of Stress Analysis

A typical lane width was investigated individually for dead, live and temperature loadings. Separate analyses were done for each of the six AASHTO live loadings H20-44, (both truck and lane), legal Type 3, legal Type 3-3, legal Type 3-S2 as well as for the orginal design live load of 125 psf (625 kg/sq. m). Temperature differentials of both plus and minus 30° F were considered to determine temperature stresses. Such temperature variations were those utilized in the original design and are believed to be reasonable values for this massive structure.

The resulting stresses were then combined into AASHTO loading Groups I and IV together with appropriate live load impact factors. These twenty-one loading combinations together with corresponding allowable stress factors are shown on Exhibit 1, Loading Combination for Maximum Stresses in Concrete Arches.

In order to ascertain the maximum live load stresses for the various truck loads, six or seven locations within a typical span were assumed for each of the five types of truck loading and the actual stresses then determined for each loading and position. These "Trial Loading Positions for Determination of Maximum Live Load Stresses" are shown on Exhibit 2.

A computer model with twenty-two discrete elements was established for the purpose of finite element analysis of the typical arch structure. The original design drawing were used to obtain the dimensional characteristics of this model. The Theoretical Model of Typical Arch is shown in Exhibit 3.

Since the in-depth inspection had found that the exterior arch ribs were often seriously corroded the presence of these ribs was neglected for the purpose of analysis. As a result, the section properties of the arch model were computed on the basis of a plain concrete section, a conservative assumption. Furthermore, because of the questionable end fixity of arch structures, the arch model for all loading cases was analyzed assuming fixed as well as hinged end conditions. The upper and lower boundary solutions for the arch behavior were thus obtained for each of the loading cases and combinations as discussed earlier.

Results of Stress Analysis

The "Envelope of Maximum Stresses for Combined Loadings" based on fixed end arch condition are shown on Exhibit 4. These six envelopes were then combined to produce an absolute maximum stress envelope for this end condition as shown on Exhibit 5. Similar values and stress envelopes were developed for hinged end arch condition and are shown on Exhibits 6 and 7.

When stresses shown on Exhibit 8 were adjusted for an allowable stress factor of 1.25 for D+L+I+T, the maximum values for required concrete strength were 620 psi (43 kg/sq. m) compression and 345 psi (14 kg/sq. m) tension, under the assumption of full end fixity. Under the action of these full high tensile stresses, the assumed unreinforced concrete arch will tend to crack at those locations and then behave as a ture two hinged arch producing negligible tensile stresses and a maximum of 540 psi (38 kg/sq.m) compression stress. It was believed, however, that the presence of steel ribs inside the concrete arch would result in partial end fixity. The stresses produced from this end condition will be somewhere between 540 (38 kg/sq.m) and 620 psi (43 Kg/sq,M)

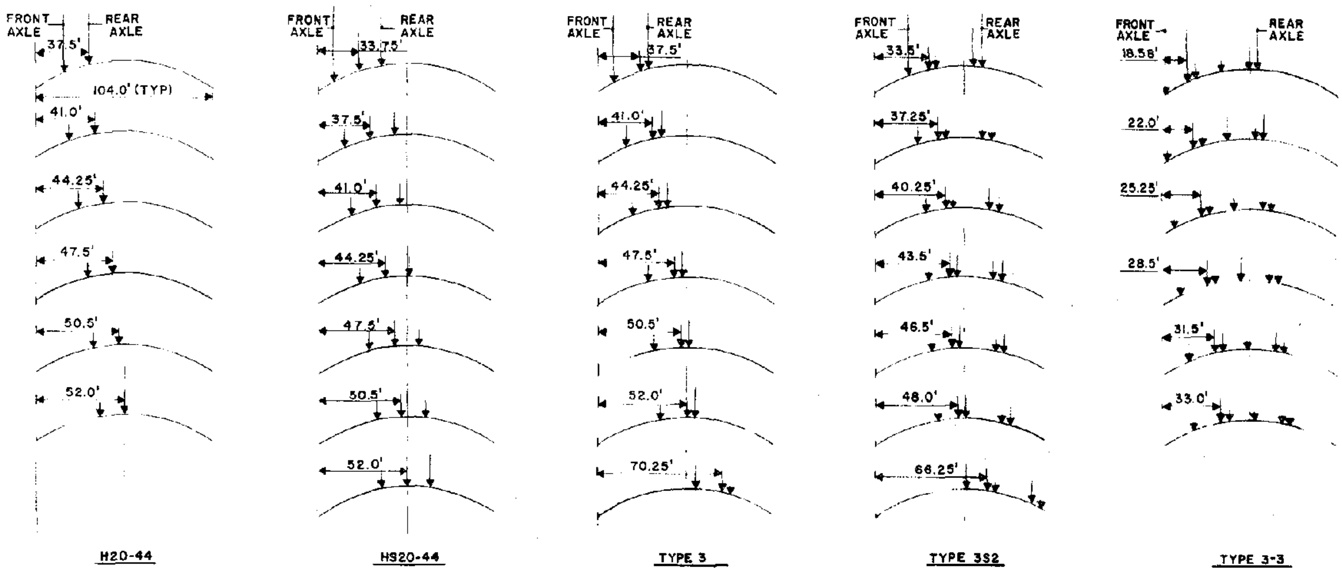
From above, it was concluded that a concrete strength of about 600 psi (42 kg/sq,cm) would enable the arches to very adequately support any of the indicated loadings.

LOADING COMBINATIONS FOR MAXIMUM STRESSES IN CONCRETE ARCHES

NATURE OF LIVE LOAD	AASHTO LOADING GROUP *	LOAD FACTORS (INCLUDES IMPACT WHERE APPLICABLE)										ALLOWABLE STRESS FACTOR
		DEAD	H20-44	HS20-44	H20-44 LANE	TYPE 3	TYPE 3-3	TYPE 3S2	TEMP+30°F	TEMP-30°F	125 PSF	
H20-44	I	1.0	1.22	0	0	0	0	0	0	0	0	1.00
	IVa	1.0	1.22	0	0	0	0	0	1.0	0	0	1.25
	IVb	1.0	1.22	0	0	0	0	0	0	1.0	0	1.25
HS20-44	I	1.0	0	1.22	0	0	0	0	0	0	0	1.00
	IVa	1.0	0	1.22	0	0	0	0	1.0	0	0	1.25
	IVb	1.0	0	1.22	0	0	0	0	0	1.0	0	1.25
H20-44 LANE	I	1.0	0	0	1.22	0	0	0	0	0	0	1.00
	IVa	1.0	0	0	1.22	0	0	0	1.0	0	0	1.25
	IVb	1.0	0	0	1.22	0	0	0	0	1.0	0	1.25
TYPE 3	I	1.0	0	0	0	1.22	0	0	0	0	0	1.00
	IVa	1.0	0	0	0	1.22	0	0	1.0	0	0	1.25
	IVb	1.0	0	0	0	1.22	0	0	0	1.0	0	1.25
TYPE 3-3	I	1.0	0	0	0	0	1.22	0	0	0	0	1.00
	IVa	1.0	0	0	0	0	1.22	0	1.0	0	0	1.25
	IVb	1.0	0	0	0	0	1.22	0	0	1.0	0	1.25
TYPE 3S2	I	1.0	0	0	0	0	0	1.22	0	0	0	1.00
	IVa	1.0	0	0	0	0	0	1.22	1.0	0	0	1.25
	IVb	1.0	0	0	0	0	0	1.22	0	1.0	0	1.25
ORIGINAL 125 PSF DESIGN LOAD	I	1.0	0	0	0	0	0	0	0	0	1.0	1.00
	IVa	1.0	0	0	0	0	0	0	1.0	0	1.0	1.25
	IVb	1.0	0	0	0	0	0	0	0	1.0	1.0	1.25

* Loading group V, because of the higher allowable stress factor (1.40), never resulted in critical stress levels.

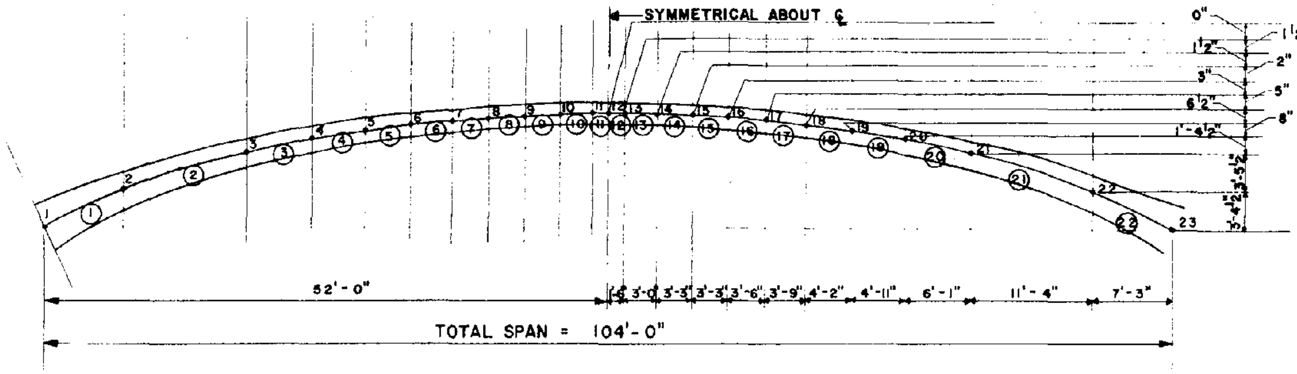
EXHIBIT 1



TRUCK LOADINGS

EXHIBIT 2

TRIAL LOADING POSITIONS FOR DETERMINATION OF MAXIMUM LIVE LOAD STRESSES

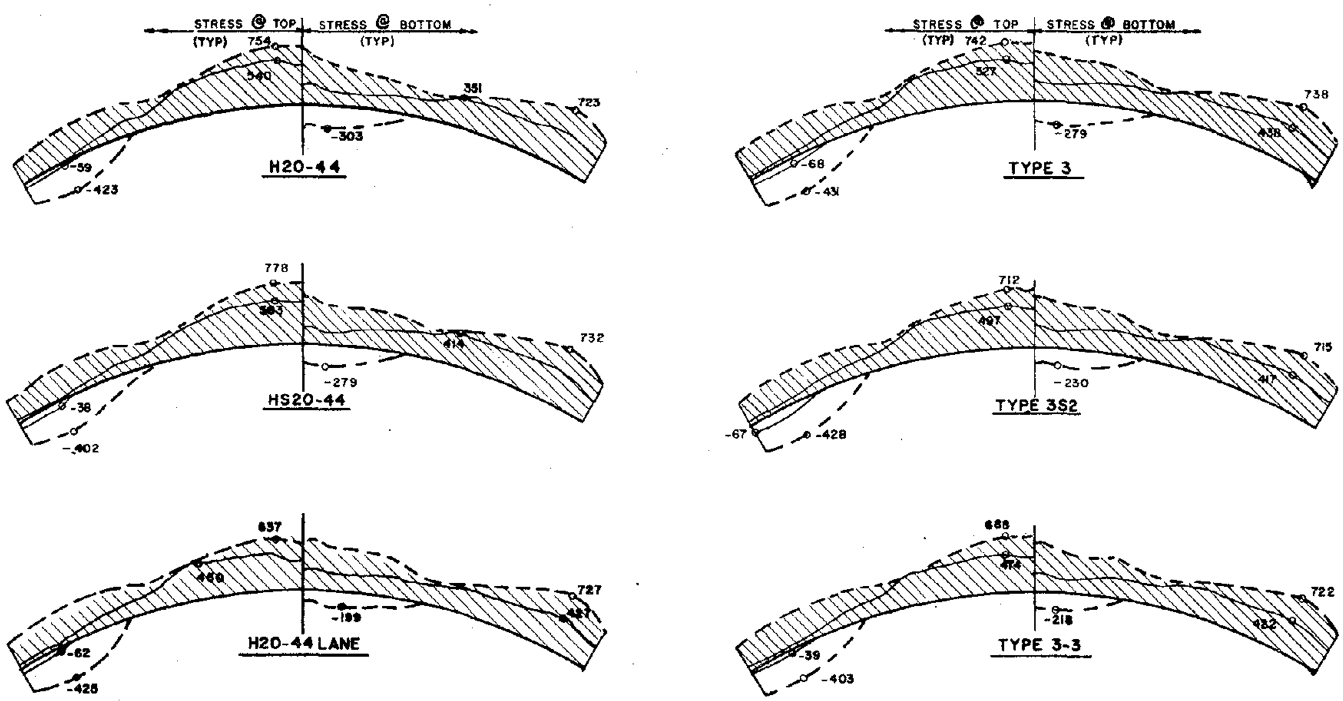


THEORETICAL MODEL OF TYPICAL ARCH
Scale: 1/8"=1'-0"

NOTE: All arch analysis has been premised upon the assumption of two possible end or support conditions:

Assumption A: Fixed ends
Assumption B: Hinged ends

EXHIBIT 3



LEGEND
COMPRESSIVE STRESS
TENSILE STRESS
— D+L+I LOADS
-- D+L+I+T LOADS

NOTE: Stresses due to a uniform 125 psf loading (basis of original bridge design in 1905 *) are similar to those for H20-44 lane loading.

EXHIBIT 4

ENVELOPE OF MAXIMUM STRESSES FOR COMBINED LOADINGS
(ASSUMPTION A - ARCH ENDS FIXED)

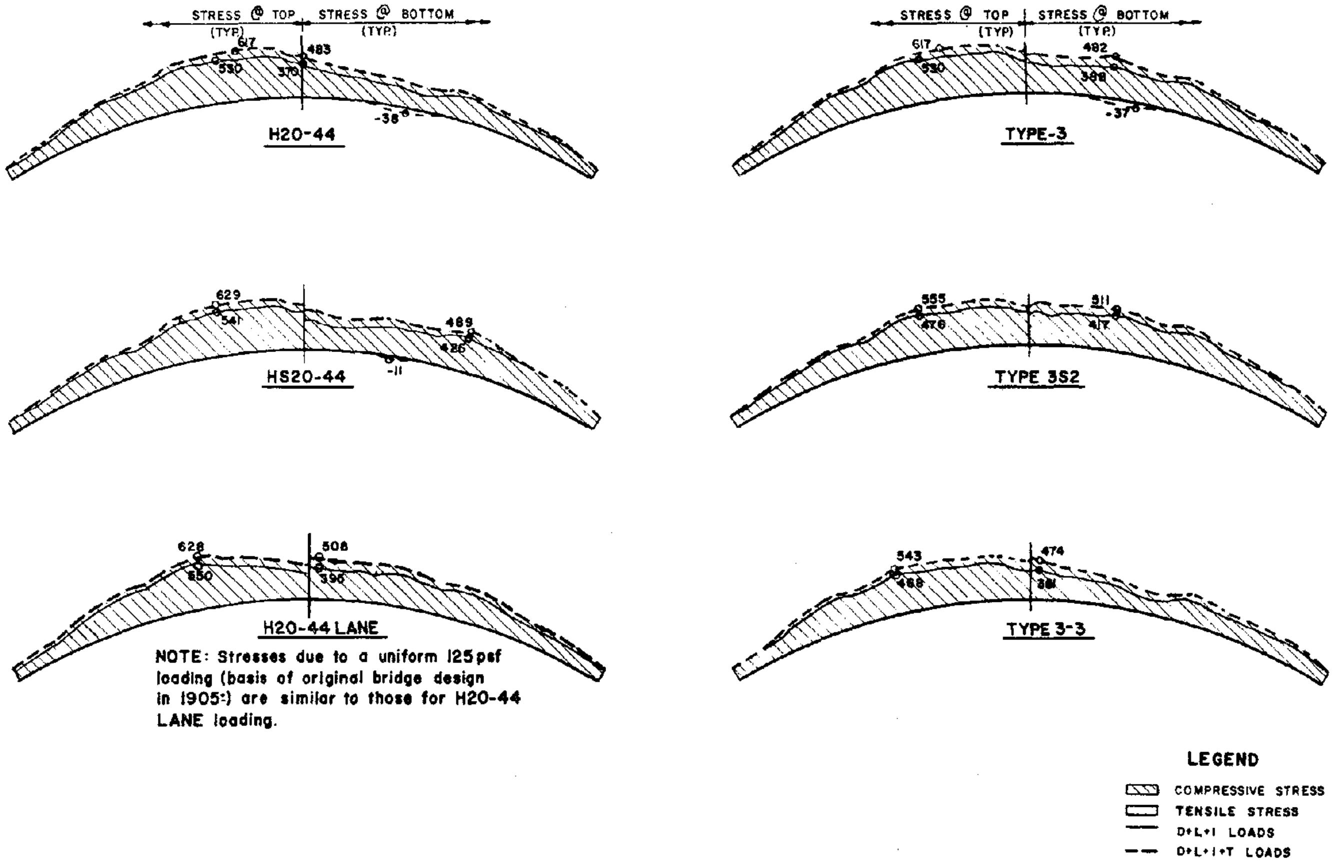


EXHIBIT 5

ENVELOPE OF MAXIMUM STRESSES FOR COMBINED LOADINGS
(ASSUMPTION B-ARCH ENDS HINGED)

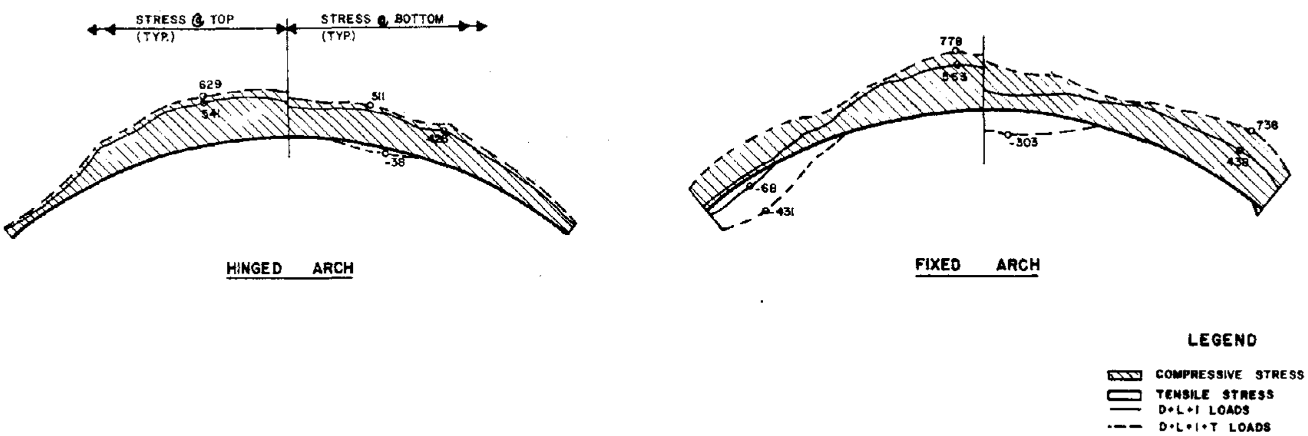


EXHIBIT 6

ENVELOPE OF MAXIMUM STRESSES FOR DEAD, LIVE AND TEMPERATURE LOADING COMBINATIONS DEFINED ON EXHIBIT A-64

CONCRETE QUALITY

To determine the general quality of the concrete in the structure, a number of test were performed on specimens taken from the hardened concrete by core drilling. These test were the compressive strength of the concrete, petrographic analysis, sodium chloride contents, percentage loss in standard freeze-thaw tests and air entrainment. A total of 36 core samples were taken from various locations in the six concrete arch spans.

In order to test the compressive strength of the concrete, a reasonably sound length of concrete core is required. Consequently, the compression tests were all made on the better concrete. The strengths determined from these tests ignore the portions of the core that were either excessively honeycombed, or were in a rubble condition or where no recovery was possible. There were 18 concrete compressive strength tests. Of these, 8 were on sound concrete, 6 on good concrete, 1 on fair concrete and 3 on poor concrete.

The best measure of concrete quality is the concrete compressive strength and the quality designation, determined from the petrographic analysis. The quality designation is consistently sound to good for high strengths and fair to poor for low strengths. Of the 66 quality evaluations, 18 were sound, 20 were good, 11 were fair and 17 were poor. It can be assumed that the concrete strengths reasonably represent the sound, good and fair quality concrete but they do not represent the poor quality concrete since it was seldom possible to test the poor concrete.

The quality evaluations had a ratio of (sound + good + fair) to (poor) of 49 to 17 or 2.88 while the same ratio for strength tests performed was 15 to 3 or 5.0. From this, it was concluded that the concrete strength evaluation must be adjusted downward so that it will represent all of the concrete in the structure. If the result of the compressive strength tests analysis was multiplied by

$$\frac{(1 - 2.88/5.0)}{2.88/5.0} \times 100 = 73.6\%,$$

it would be corrected for the poor concrete reported in the quality evaluation findings not represented by the compressive strength tests.

The percentage loss as measured in a standard freeze-thaw tests is also a measure of concrete quality. A total of 18 of these tests were made. These tests confirm the variable quality of the existing concrete which was indicative of poor control during construction. Ten of the eighteen tests had excessive weight loss. This was indicative of concrete without air entrainment and also of the low strength concrete. The air entrainment test results showed that air entrained concrete was not used and that the only air present was entrapped air. The results of the freeze-thaw tests confirmed that the existing concrete had a low durability and that if extended life was desired, the concrete should be protected, as much as possible, from the action of freezing and thawing. This conclusion reinforced the need for a composite protective concrete cap across the top surfaces of the arches and substantial repair to the bottom surfaces.

Eighteeen tests were performed to determine the sodium chloride content. Except for one test, all results were within or close to an acceptable value of 0.25 lbs./c.y. maximum. It was concluded that penetration of sodium chloride either from the roadway surface or from salt spray from the water below, was nota problem for this structure. The corrective measures being recommended would also reduce the possibility of excessive sodium chloride penetration in the future.

Perhaps the most consistent and predominate feature determined from the petrographic analysis was the number of horizontal cracks present in the test specimens. This cracking was not restricted to fair and poor quality concrete but was also prevalent in the sound and good quality concrete. This cracking indicated that a conservative approach should be utilized in the repair and upgrading of the concrete structure.

There was no indication that the different types of coarse aggregate, either crushed coarse grained granite (CCGG) or crushed dolomite limestone (CDL) had any affect on the concrete quality.

As a conclusion to the analysis of the test results of the tests taken to determine the concrete quality, it may generally be stated that the quality of the concrete varied throughout the structure by the following percentages:

Quality	%
Sound	27
Good	30
Fair	17
Poor	26
	100

Seventy-four percent of this concrete was considered acceptable and it was feasible to repair and upgrade the concrete arch structures.

CONCRETE STRENGTH EVALUATION

In order to assess the load carrying capacity of the concrete arch spans, a determination of the concrete compressive strength was required. To provide the basic data for this determination, specimens taken from the hardened concrete were obtained by core drilling and then compression tests were made on samples prepared from these test specimens in the laboratory.

The design concrete strength was determined by use of the procedures provided in the American Concrete Institute Standard "Recommended Practice for Evaluation of Compression Tests Results of Field Concrete" (ACI 214). In this procedure, the compression test results of field concrete were obtained by testing the samples prepared from core taken from the hardened concrete. The design concrete strength determined by this procedure represented a design strength of concrete over 70 years old.

All design specifications are based on concrete with a compressive strength at the age of 28 days. The increase of strength with age is part of the available factor of safety.

Hence, to properly compare the required strength of the concrete from the result of stress analysis, discussed earlier, with the actual strength of the concrete available in the field; it was necessary to correct the seventy year old concrete design strength to a 28 day age.

Seventy Year Old Design Strength

Eighteen compression tests were made on specimens taken from the concrete arches. These test results were converted to the standard height to diameter ratio for evaluation and the results were as follows:

Sample No.	Comp. Strength X psi	(kg/sq.cm)
1A	5610	(393)
1C	1915	(134)
1D	6890	(482)
2C	6690	(468)
2D	6080	(426)
2F	1910	(134)
3A	4735	(331)
3D	5415	(379)
3E	6190	(433)
5A	4715	(330)
5C	5080	(356)
5D	4935	(345)
6B	3790	(265)
6C	4830	(338)
6D	5420	(379)
7C	1650	(116)
7D	4330	(303)
7F	2590	(181)
TOTAL	82775	

Average Strength = $\bar{X}$ = 4,600 psi (322 kg/sq.cm)

Utilizing ACI 214 for determination of the design concrete strength, if the Standard Deviation was denoted by σ, then:

$$\sigma = \sqrt{\frac{(X_1 - \bar{X})^2 + (X_2 - \bar{X})^2 + \ldots (X_n - \bar{X})^2}{n}}$$

Where X_1, $X_2 \ldots X_n$ were the values of compressive strength for the individual test and n was the number of tests, in this case equal to 18.

σ = 1582 psi (111 kg/sq.cm)

The coefficient of variation, V, was determined by use of ACI 214 is:

$$V = \frac{\sigma}{\bar{X}} \times 100 = 34.4\%$$

From this value it was stated that the Standard of Concrete Control was "Poor" per ACI 214.

The concrete design strength f'_c, before any age correction was calculated by:

$$f'_c = \bar{X} \quad (1 - tV)$$

Where $\bar{X}$ and V were the average compressive strength and the coefficient of variation expressed as a decimal, respectivel. In this equation, t was a constant depending upon the proportion of tests that may fall below f'_c and the number of samples used to establish V. A table listing values of t for these two variables is provided in ACI 214. From this table, assuming 4 out of 18 samples fell below the value of f'_c, the value of t was calculated as 0.792.

Then:

f'_c = 3345 psi (234 kg/sq.cm)

Correction for 28 Day Strength

As stated before, this value of design strength was for 70 year old concrete. It had to be corrected to a 28 day age value. Usual procedures for relating concrete strengths at different ages involved use of the maturity values for the various ages. The maturity is the product of hours and degrees Fahrenheit for all temperatures above a minimum value. For seventy year old concrete the procedure was invalid because the concrete could not gain any strength once all the available cement had been hydrated.

However, records of tests at different ages show that concrete continues to gain strength over a period of 4 to 5 years. The increase for the first year alone ranges from 22% to 53%. If the age correction was calculated based on a 4 year limit, the maturity correction procedure could be utilized. Probably the most valid procedure is that developed by J.M. Plowman (March, 1956).

From this article the following relationship has been taken:

$$\%\text{ strength at } 35600^{\circ}\text{F. hr.} = A + B \log_{10} \frac{(\text{Maturity})}{(10^3)}$$

The 35600°F. hr. was the maturity of 28 day old cylinders cured by standard methods. A and B were constants which varied with the range of concrete strength under consideration. For this case, from Plowman's article:

A = -7
B = 68

Without getting involved in maturity calculations it was assumed that all of the cement had been hydrated and that the value of the term

$$\log_{10} \frac{(\text{Maturity})}{(10^3)} \text{ equals } 1.0$$

With this assumption then:

% strength at
35600°F. hr. = 61%

or

f'_c at 28 days = 2040 psi (143 kg/sq.cm)

This result appeared to be logical because most concrete strengths in the early 1900's were around 2000 psi (140 kg/sq.cm). This was ascertained from old textbooks such as Taylor, Thompson and Smulski's "Concrete Plain and Reinforced" which makes a statement to this fact. For this reason and based on the above analysis, the design strength of the concrete in the existing structure was established as 2000 psi (140 kg/sq.cm).

Allowable Stress

While the actual concrete strength in the structure had been established as 3345 psi (235 kg/sq.cm) and the design strength established as 2000 psi (140 kg/sq.cm), it still needed to be determined what acceptable stress level was permitted. In the early 1900's the allowable compressive stress permitted, f_c, was 0.18 f'_c. The coefficient 0.18 anticipated the probable variable quality and therefore strength of the concrete. However, in determining a design strength of f'_c = 2000 psi (140 kg/sq.cm), this variability of concrete strength was already considered. Therefore, it was recommended that a higher coefficient 0.40 (permitted by 1977 AASHTO) be used and:

$$f_c = 0.40\ f'_c = 800 \text{ psi } (56 \text{ kg/sq.cm})$$

Under the discussion of Concrete Quality it was shown that the concrete strength test results were modified by a factor of 0.73 to represent all of the concrete in the structure. This 0.73 multiplier was applied to the computed allowable stress value of 800 psi (56 kg/sq.cm). Accordingly, a basic net allowable working stress of about 600 psi (42 kg/sq.cm) was recommended as a reasonable limiting value for the existing concrete in the six arches.

Alternatively, it could be argued that it was not necessary to correct the concrete strength to a 28 day age; and the actual strength of 3345 psi (235 kg/sq.cm) should be used as f'_c. The allowable stress, using this approach, should be computed by using the coefficient of 0.18 without any further modifications for concrete quality. This value also works out to be 600 psi (42 kg/sq.cm).

Hence, it was concluded that 600 psi (42 kg/sq. cm) was a reasonable allowable working stress for existing concrete.

RATING

The stress analysis of the arches indicated that to fully support all various AASHTO loading combinations, a concrete strength of about 600 psi was required. The analysis of the cores indicated that the existing concrete can develop the 600 psi required strength. Hence, it was concluded that the arches when rehabilitated will adequately support the various loading combinations. Inventory and operating ratings for each loading combination were then computed as follows:

Loading/Vehicle Wt.		Inventory Rtg.		Operating Rtg.	
HS 20	(36T)	45T	(41MT)	68T	(62MT)
H20	(20T)	26T	(24MT)	39T	(35MT)
3	(25T)	30T	(27MT)	42T	(38MT)
353	(36T)	43T	(39MT)	60T	(55MT)
3-3	(40T)	92T	(84MT)	151T	(137MT)

ACKNOWLEDGEMENTS

The author wishes to acknowledge the valuable assistance provided by the clients of this project; New York State and New York City Department of Transportations. The author would also like to acknowledge the valuable contributions provided in the development of concrete evaluation by Mr. Jerome S.B. Iffland.

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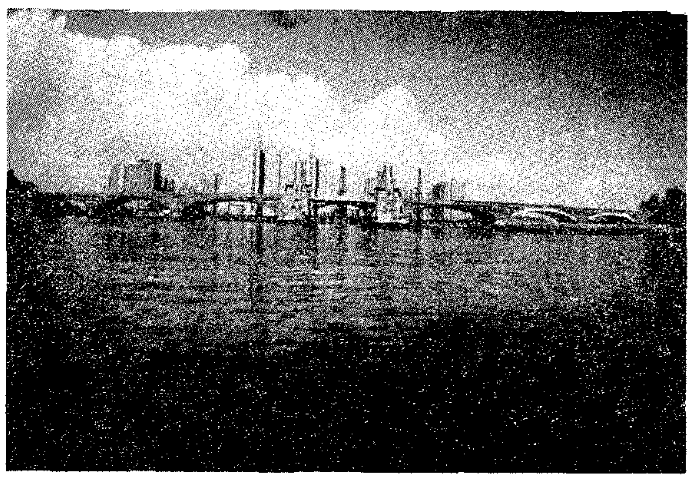
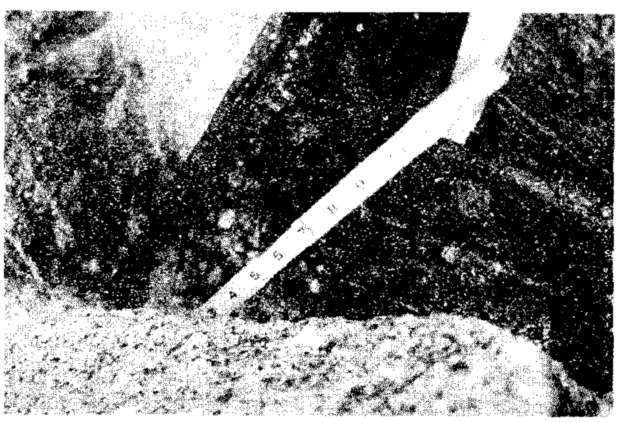


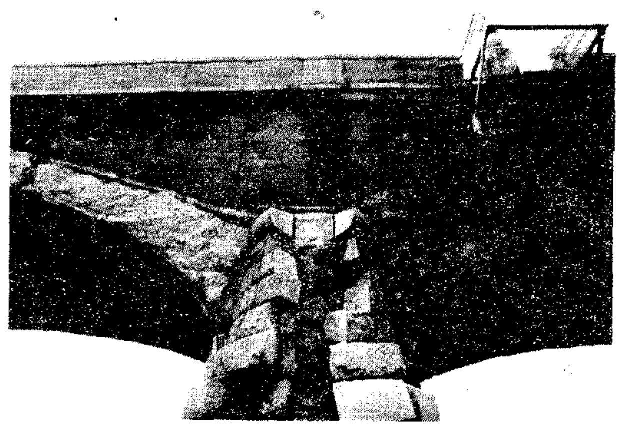
BRIDGE ELEVATION

BRIDGE POSTING

UNDERSIDE OF ARCH AT PIER IV



ARCH RIB DETERIORATION AT PIER IV



PIER V

INVESTIGATION INTO SAFETY AND RATING OF THROUGH TRUSS BRIDGES–TWO CASE STUDIES

Ramesh R. Patel

Project Manager,
Sheladia Associates, Inc., Riverdale, Maryland

SUMMARY

Recent estimates of deficient bridges indicate that, nationally, there are over 7,000 bridges classified structurally or geometrically deficient and 28,000 functionally obsolete. Like most other states, West Virginia, the second leading coal producer in the U.S.A., is also faced with the problems of maintaining and upgrading old highway structures. During this last decade, the increased threat of an energy crisis has intensified coal consumption. Consequently, the increased heavy traffic on country roads and highways became a major concern of the West Virginia Department of Highways resulting in an investigation into the safety and rating of all bridges.

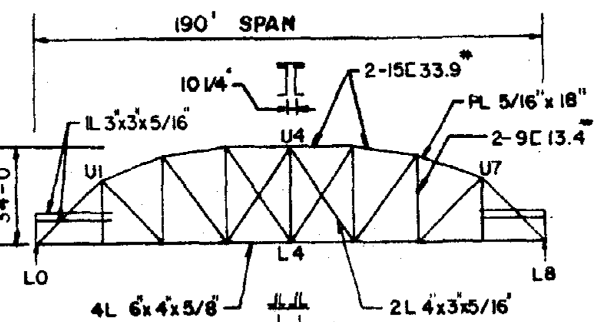
Most of the bridges built during the early of this century are truss brdiges of spans up to 200 feet. Most of them also have narrow roadways. The field inspection of Ellamore and Harmon truss bridges revealed that the lack of periodic inspection and poor maintenance were the major factors causing many unsafe structural deficient bridges.

INTRODUCTION

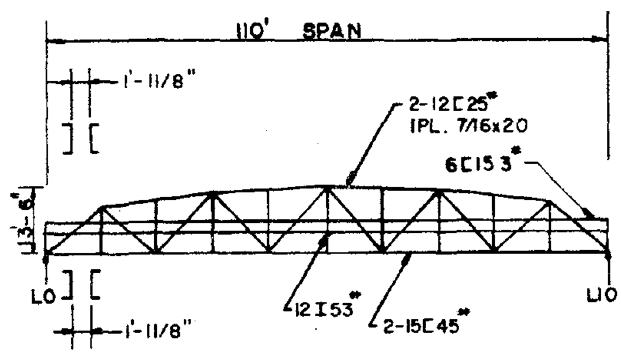
This paper presents the result of an inspection made to ascertain the physical condition of Ellamore and Harmon bridge structures and summarizes the investigations and conclusions relative to their load carrying capacities.

Ellamore and Harmon truss bridges are located on U.S. Route 33 in Randolph County. The Ellamore bridge structure over the Middle Fork River consists of 190 feet simple span riveted steel thru truss and 20 feet wide two lanes of reinforced concrete deck supported on full height concrete abutments. Likewise, the Harmon bridge over the Dry Fork River is a 110 feet simple span riveted steel pony truss with 24 feet wide two lanes of reinforced concrete deck. The Ellamore and Harmon bridges were built in 1931 and 1933 respectively. During this time, thru (with top lateral and portal bracings) and half thru or pony truss (without top lateral system) bridges were more commonly built in the state of West Virginia.

The floor deck system of both bridges consists of reinforced concrete deck slab without sidewalks on steel stringers and floor beams. The substructures are consisting of full height concrete abutments with spread footings.



ELLAMORE BRIDGE TRUSS
Fig. 1



HARMAN BRIDGE TRUSS
Fig. 1A

INSPECTION AND ASSESSMENT

The field inspection was conducted during the period of May 19 thru May 22, 1981. In-depth observations and measurements of various elements were made to verify the previous inspection performed by the West Virginia Department of Highways during September-October of 1978 to detect the presence of distress and to determine the extent of deterioration which might affect the operational integrity of the bridges. Photographs were taken to show the general layout of the bridge and to depict deteriorated conditions. Because of the common physical characteristics of both bridge structures, the following detailed comments on various portions are presented:

a. Alignment

The Ellamore structure is on tangent section of the roadway and is not skewed. The horizontal curve starts beyond the west abutment and a vertical sag curve begins after the east abutment. Sight distance is fair to poor.

The Harmon bridge is located on a curved section of the roadway and is not skewed. The junction of Rand Co. 20 (L.S.) is located about 300 feet from the west abutment. The sight distance is adequate.

b. Approaches

The approaches were in fair condition with some surface cracks on bituminous pavement. The sliding plate expansion joint openings were measured for both bridge structures. The measurement was taken with respect to the distance from the end of the truss to the face of the backwall at both abutments. There were no approach guardrail or hazard warning paddles.

c. Waterway

The waterway appeared to be causing no problems for either bridge. The stream bed could be seen through the water. Channel profiles were taken along the upstream and downstream of the structures and were measured with respect to bottom chord at every panel point of truss. There were no indications of detrimental scouring or under-mining porblems due to the fact that none of the abutments were located in the normal stream channel flow.

d. Substructure

The full height abutments of Ellamore bridge were in fair condition. The seat concrete exhibited medium scaling around the bearing areas. The west abutment stem had two horizontal cracks which extended through the wingwalls. These cracks may have been construction cold joints located at 6 feet and 11 feet from the seat. The abutment stems had surface cracks. The downstream wingwall of west abutment exhibited cracks with efflorescent and spalling concrete. The upstream wingwall of east abutment also had three horizontal cracks and spalling concrete. A large accumulation of debris was found on the seat of east abutment.

The abutments of Harmon bridge were also in fair condition. The backwall and the seat concrete of west abutment was spalling and dis- integrating at upstream rocker bearing. The corners of seat at both rocker bearings of east abutment were broken and spalling. There was heavy accumulation of debris on the east abutment seat.

In general, the face of abutments and wingwalls exhibited excessive surface cracking, efflorescence and weathering of the concrete. There were no weep holes in these substructures.

e. Superstructure

The superstructure of a truss bridge usually consists of the following elements:

1. The deck
2. Stringers--longitudinal beams to support the deck
3. Floor Beams--transverse beams that support the stringers
4. Trusses that support the floor beams.

1. The deck: The reinforced concrete deck slabs of both bridges were in very poor condition. Most of the deck surface was cracked and exhibited efflorescence. The curb was spalling and rebars were exposed on both sides of the structure for the entire length of the bridge. All down spouts were completely deteriorated and blocked with debris, hence the drainage system on the bridge was in poor condition.

ELAMORE BRIDGE
DETERIORATED CURB

HARMON BRIDGE
DETERIORATED CURB

2. Stringers: Because of the poor drainage system and broken curbs, the exterior stringers of Ellamore and Harmon bridge structures were badly corroded throughout their entire spans. The estimated section loss was up to 50%. Connection angles and rivet heads were badly corroded. The top flange of stringers exhibited rusting in the area of deteriorated and corroded deck.

3. Floor Beams: The largest amount of section loss in the floor beams was observed at end connections that had been exposed to de-icing chemicals. This deterioration occurred due to the poor deck and curb condition. The percentage section loss is shown in Figures 2 and 3.

4. Trusses: Varying amounts of corrosion were noted on all trusses of Ellamore and Harmon bridge structures. The Harmon bridge superstructure was found to be painted after the previous inspection of September 1978. In general, the largest amount of section loss in the trusses was observed along the bottom chord. All connections, gusset plates, splice plates, lower lateral connection plates and many of the rivet heads had corroded heavily with section loss of 10 to 80%.

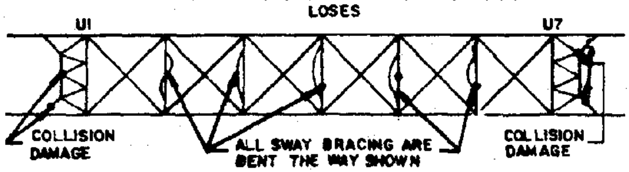
   The top chords and upper connections were in good condition. There was some surface rust. The rivets exhibited some surface corrosion. The verticals and diagonal members exhibited pitted areas with section loss up to 25%. The bottom lateral bracing angles were in poor condition. Horizontal legs of angles, connection plates and rivet heads were reduced up to 80%.

   The portal frame at both abutments of Ellamore bridge exhibited heavy collision damage to the lower strut and knee bracings. All sway bracings, also had some collision damage.

   The bearings were in fair to good condition. There was some light to medium rust and pitting due to deck drainage problem and debris build up.

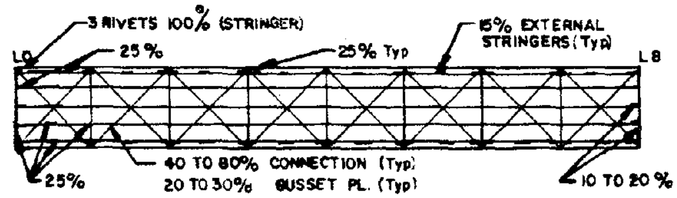
LOCATION OF DETERIORATION

PERCENTAGES SHOWN INDICATE SECTION LOSES



TOP LATERAL BRACING

Fig. 2

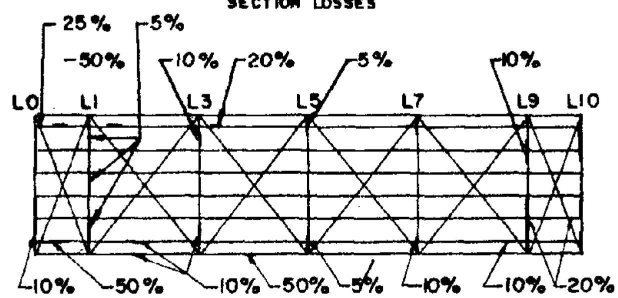


DECK FRAMING PLAN

Fig. 2A

LOCATION OF DETERIORATION

PERCENTAGES SHOWN INDICATE SECTION LOSSES



DECK FRAMING PLAN

Fig. 3

STRUCTURAL RATING

The current system of live loading for the design of highway structures is that of the American Association of State Highway and Transportation Officials (AASHTO). Two systems of loading are specified, the H and the HS. Ellamore and Harmon bridges were originally designed for loading equivalent to H15-44, a 15 ton truck load.

The rating analysis was made in accordance with 1977 AASHTO Standard Specifications for Bridges--Twelfth Edition and 1978 AASHTO Manual for Maintenance Inspection of Bridges. The bridges were rated at two load levels by working stress method which accounts for the strength of the materials of construction in their current state.

At the first or upper load level, the capacity rating is to be referred to the Operating Rating. The Operating Rating is the absolute maximum permissible load level to which the structure may be subjected. At the second or lower load level, the capacity rating is to be referred to as the Inventory Rating. The Inventory Rating is a load level which can safely utilize an existing structure for an indefinite period of time. The ratings are expressed as follows:

Operating Rating, $$H_O = \frac{f_O - f_{D.L.}}{f_{LL+I\ (H15)}} \times H15$$

Inventory Rating $$H_I = \frac{f_I - f_{D.L.}}{f_{LL+I\ (H15)}} \times H15$$

or

$$H_I = \frac{f_{SR}}{f_{LL+I}} \times H15$$

where H_O = Operating Rating load in tons

H_I = Inventory Rating load in tons

f_O = Allowable stress in ksi - Operating rating

f_I = Allowable stress in ksi - Inventory rating

f_{DL} = Dead load stress in ksi

f_{LL+Z} = Live load + impact stress in ksi (H15-44 loading)

f_{SR} = Allowable Range of stress in ksi--allowable fatigue stress.

H15 = H15-44, a 15 ton truck load

For unknown structural steel used during 1905 to 1936, an allowable operating stress of 22.5 ksi and an allowable inventory stress of 16.0 ksi were used in the analysis.

The following is a summary of a structural rating:

1. Ellamore Bridge:

OPERATING

Truss Member (upstream U_3U_4)	25 tons
Floor Beam (@ L_1)	43 tons
Stringers--Min. Lateral Support (Exterior)	19 tons

INVENTORY

Truss Member (upstream (U_3 U_4)	14 tons
Floor Beam (@ L_1)	14 tons
Stringers--Min. Lateral Support (Exterior)	5.4 tons

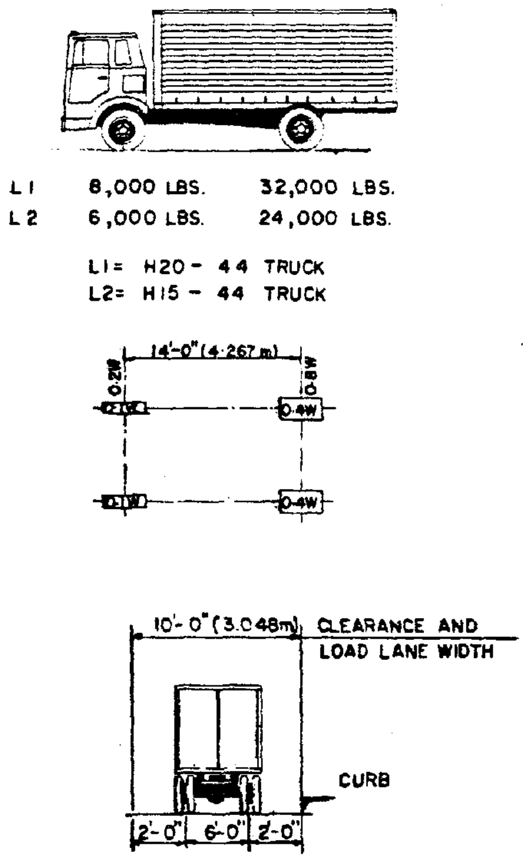
2. Harmon Bridge:

OPERATING

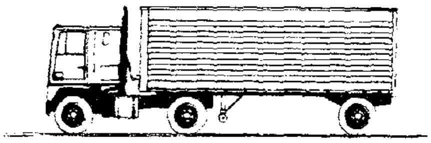
Truss Member (upstream U_4U_5)	44 tons
Floor Beam (@ L_O)	41 tons
Stringers--Min. Lateral Support (L_O L_1)	17 tons

INVENTORY

Truss Member (upstream U_4U_5)	9 tons
Floor Beam (@ L_O)	12 tons
Stringers--Min. Lateral Support (L_1 L_2)	5 tons

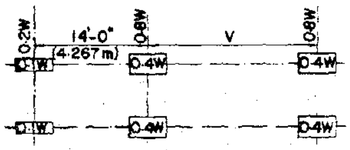


STANDARD H (M) TRUCK



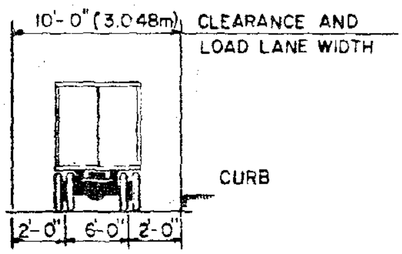
L1 8,000 LBS 32,000 LBS 32,000 LBS
L2 6,000 LBS 24,000 LBS 24,000LBS

L1 = HS20-44 TRUCK
L2 = HS15-44 TRUCK



W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H(M) TRUCK.

V = VARIABLE SPACING-14 FEET TO 30 FEET (4.267 to 9.144m) INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES.



STANDARD HS (MS) TRUCK

RECOMMENDATIONS:

The following recommendations were implemented to ·nsure structural integrity and increase load capacity to ·rry H-20 (AASHTO Loading) loading.

1. To halt deteriorated condition and ensure structural integrity of the substructure, all concrete surfaces should be repaired with epoxy bonded grout or pneumatically applied mortar.

2. Provide lighter bridge grating deck, thus reducing the dead load forces.

3. All stringers should be replaced and floor beams be strengthened or replaced.

4. The damaged portal frames and sway bracings of the Ellamore bridge should be replaced and that vertical clearance be improved.

5. Significantly deteriorated members of truss to be repaired.

6. To post the load limit signs.

7. To clean and paint bridge structures.

REFERENCES:

Standard specifications for Highway Bridges adopted by the American Association of State Highway and Transportation Officials (AASHTO). Twelfth Edition - 1977.

1978 AASHTO Manual for Maintenance Inspection of Bridges.

Bridge Inspection and Structural Analysis by Sung H. Park.

Bridge Inspector's Training Manual 70, 1979, U.S. Department of Transportation/Federal Highway Administration.

Bridge Inspection Manual, Developed by West Virginia Department of Highways.