

Rehabilitation, Renovation, and Preservation of Concrete and Masonry Structures

Gajanan Sabnis — Editor



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PREFACE

This volume presents papers which were prepared for three symposia sponsored by the Technical Activities Committee of the American Concrete Institute. The symposia were held at the ACI Conventions in Quebec City, Canada, September 1981; and Atlanta, Georgia, January 1982.

The symposia were organized by an ad hoc committee appointed by TAC. The chairman was I. Leon Glassgold; and other members were Howard Newlon, Jr.; Peter Smith; Emory L. Kemp; W. Gene Corley; and Joseph A. Dobrowolski. In September 1981, the permanent Committee 364 on rehabilitation was established. This committee completed the compilation of the present volume.

The rehabilitation of concrete and masonry structures is of growing importance for both historical and economic reasons. This volume provides a current view of the subject, and emphasizes the need for further research and education. A prerequisite to undertaking any rehabilitation project is to determine the structural integrity of the frame, its components, and its foundations. Dov Kaminetzky describes the various steps of this procedure.

An overview of the essential steps in modifying old structures for new use is presented by Boris Bresler. Edward Cohen, Norval Dobbs, and William Combs describe the rehabilitation of the Kresge Auditorium at MIT. R. L. Nene discusses the effectiveness of shotcreting in the repair and restoration of reinforced concrete columns.

The evaluation of existing materials and selection of repair materials are critical steps in rehabilitation. James Clifton's description of nondestructive evaluation methods is based on their principles of operation, information they provide, applications, expertise required to use them, and advantages and limitations of their use. Stella Marusin presents an example of the restoration of a reinforced concrete frame with acrylic modified portland cement concrete and epoxy mortar. Evaluation and selection of repair materials and techniques are discussed further by Theodore Prudon and Jerry Stockbridge in their case history of the Woolworth Building facade renovation. The use of epoxy mortar in subzero replacement of defective grout is described by Floyd Dimmick.

Zareh Gregorian discusses the investigation, renovation, and restoration of ancient brick and stone masonry, and modern reinforced brick masonry. James Gross provides an overview of the regulatory constraints for rehabilitation and identifies the need for improved evaluation techniques.

Water-retaining concrete structures pose a particularly difficult yet common rehabilitation project. Stephen Tsui provides a case study of the rehabilitation of a clarifier structure.

Cathodic protection is an effective means to prevent steel corrosion and subsequent concrete deterioration, and Kenneth Hover shows how it was used for maintenance of a reinforced concrete parking garage.

Committee 364 would like to thank all authors who contributed papers, and gratefully acknowledges the time and effort of committee members who reviewed them per ACI procedure.

Gajanan M. Sabnis
Symposium Chairman
Committee 364

CONTENTS

PREFACE.....	111
ESSENTIAL STEPS IN ADAPTATION OF OLD BUILDINGS	
By Boris Bresler.....	1
NONDESTRUCTIVE EVALUATION IN REHABILITATION AND PRESERVATION OF CONCRETE AND MASONRY MATERIALS	
By James Clifton.....	19
INSPECTION, ANALYSIS, AND RESTORATION OF MIT KRESGE AUDITORIUM	
By Edward Cohen, Norval Dobbs, and William Combs.....	31
EVALUATION, REHABILITATION, AND INNOVATIVE DESIGN PROCEDURES FOR MASONRY STRUCTURES--CASE STUDIES	
By Zareh Gregorian.....	95
IMPROVING BUILDING REGULATIONS FOR REHABILITATION	
By James Gross.....	121
VERIFICATION OF STRUCTURAL ADEQUACY	
By Dov Kaminetzky.....	135
REPAIRS OF CONCRETE COLUMNS, SPANDRELS, AND BALCONIES ON A HIGH-RISE HOUSING COMPLEX IN CHICAGO	
By Stella Marusin.....	157
CATHODIC PROTECTION FOR REINFORCED CONCRETE STRUCTURES	
By Kenneth Hover.....	175
RENOVATION OF THE FACADE OF THE WOOLWORTH BUILDING	
By Theodore Prudon and Jerry Stockbridge.....	209
RESTORATION OF A CONCRETE CLARIFIER STRUCTURE	
By Stephen Tsui.....	229
SUBZERO DEGREE FARENHEIT TEMPERATURE REPAIRS OF CONCRETE WITH EPOXY MORTAR	
By Floyd Dimmick.....	239
REPAIRS AND RESTORATION OF REINFORCED CONCRETE COLUMNS	
By R. L. Nene.....	259
SI (Metric) TABLES.....	265
INDEX.....	267

SP-85-1

Essential Steps in Adaptation of Old Buildings

By Boris Bresler

Synopsis: Essential steps in modifying structures for the purpose of adapting old buildings to new use are discussed in this paper. Eight basic steps are described: (1) preliminary site visit, (2) documentation of original design and construction, (3) identification of desired changes in the building, (4) identification of code requirements applicable to building alterations, (5) development of structural modification schemes and preliminary cost estimates, (6) verification of "as is" condition, review of adequacy of proposed modification scheme and estimated cost, (7) completion of design and construction documents, including drawings and specifications, and (8) coordination of engineering design with construction quality control and accommodation of possible need for design changes during construction.

Keywords: buildings; evaluation; renovating; repairs; strengthening; structural design.

2 Bresler

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1. INTRODUCTION

In recent years, rehabilitation and renovation of existing buildings have grown to occupy a significant share of the concrete and masonry construction market. This growth has drawn an everwidening circle of developers, architects, engineers and contractors into this recycling process. The spirit of this recycling process is intended to meet the immediate need for additional space and to enhance security, stability, and beauty of the urban environment, while, at the same time, conserving basic materials and energy resources.

Three other factors contribute to the wide spread of recycling of old buildings:

- (1) need for space at reasonable cost
- (2) availability of sound old buildings, and
- (3) tax incentives through depreciation and rehabilitation cost write-offs.

In an urban environment already densely built up, rehabilitation of existing space becomes an alternative to new construction, as it frequently meets the need for additional space at a reasonable cost. The old buildings have been conservatively designed and soundly constructed; only the fittest survived, the lesser contemporaries have already been torn down.

The need and the availability are necessary conditions for recycling, but they alone are not sufficient to generate the economic forces for implementation of recycling. Implementation is largely the consequence of economic incentives generated by depreciation write-off tax advantages. In the 1976 and 1978 tax reforms, Congress extended tax incentives to the preservation of historic buildings and renovation of old buildings generally. Under these tax regulations, "old" buildings which have not been rehabilitated for at least twenty years, qualify for write-off rehabilitation expense.



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Adaptation of Old Buildings 3

The scope of the renovation project may range from relatively minor remodeling to the complete reconstruction of a building. In almost all cases, however, public safety demands a reassessment of reliable performance, both structural and non-structural performance, of the modified building.

The most challenging problems of successful renovation are:

1. Choice of the new use for the old space.
2. Adaptation of the old space for the new use at a reasonable cost.
3. Accommodation of spatial modifications with full structural safety.
4. Accommodation of building code requirements with respect to mechanical, electrical, plumbing, and fire safety requirements.

Although there are many similarities between rehabilitation and new construction, there are a number of significant differences. The essential steps in the adaptation of old buildings to new use are outlined in Table 1. Starting with a site visit and a review of proposed new use and ending with construction, the process of adaptation, from the view of structural engineering, is divided into eight steps. When rehabilitation requires only minor or cosmetic alteration, some of these steps may be omitted. In any case, the architect, the structural engineer and the contractor have to work as a team to achieve safe and economically viable schemes for the desired space modification. In this paper, however, emphasis is placed on the contributions of the structural engineer.

2. SITE VISIT

The purpose of the initial site visit is to identify the type and age of construction, overall spatial configuration, gravity and lateral load resisting systems, and to make a preliminary assessment of the present condition of the building with respect to prior damage and repair as a result of degradation due to various causes such as foundation movement, normal deterioration, corrosion, thermal and humidity variations, fire or earthquakes. At the time of the initial visit, it may not be possible to make definitive determinations of construction details and the condition of structural materials because they are often enclosed by interior or exterior finishes. In such cases, the engineer's preliminary appraisal must often rely on indirect evidence such as conventional practice and code requirements at the time the building was constructed. Nevertheless, every effort must be made to collect as much evidence as possible from visual observations and measurement of accessible structural components. Special potential problems associated with cracking, excessive deflections, inadequate fire

4 Bresler

protection, deterioration of cladding, weather tightness, and wind, snow or earthquake resistance (where applicable) should be noted. Photographic records, notes or tape recordings of observations provide an indispensable source of data for evaluation of field observations.

Subsequent to the site visit, the proposed new use of the building should be reviewed with special reference to problems related to changes in the occupancy and functional and spatial uses of the building.

3. DOCUMENTATION OF ORIGINAL DESIGN AND CONSTRUCTION

A large number of buildings considered for recycling or adaptation were used originally as industrial or commercial buildings. The "fitness" and survival of these buildings can be attributed to good design and construction workmanship. In assessing the adaptability of these buildings to new uses, it is important to identify both the original design criteria and the construction details of old buildings.

The engineer's task in evaluating an existing building is greatly simplified when drawings and specifications for original construction are available.

The important elements of the documentation which relate to original design, construction, and subsequent modifications and service life are as follows:

1. A set of drawings describing original architectural, structural, mechanical, and electrical systems. As-built and shop drawings are preferable, as these will include all the changes made during initial construction. Building permits for subsequent modifications provide valuable information on the current "as-is" conditions.
2. A set of specifications which usually form a part of the construction contract and contain information on materials, workmanship, and quality control requirements.
3. Design calculations, including specified design loads, as well as structural and foundation design.
4. Reports of foundation studies - site borings, and soil test data.
5. Inspection records reflecting construction records and notes on any deviations from the original design and/or specifications, as approved by architects or engineers, during construction.

6. Material test records which often form a part of the inspection records. These records should include data on field control tests of concrete, mill or laboratory tests of steel reinforcement and other laboratory tests.

When these are not available (as in the case of many old buildings), sketches of plans, elevations, and sections of existing buildings must be prepared by measurements obtained in the field, and material characteristics must be established by removing and testing representative samples. Special care must be exercised in locating areas where representative samples can be safely removed from the structure. The nature of the foundation for the structure must also be identified, and the general condition of the structure with reference to needed maintenance and repair should be noted.

Because drawings and specifications for the original construction greatly simplify the engineer's problem of evaluating an existing building, it is often worthwhile to expend some effort in locating such documentation. In some cases, old architectural and engineering firms have passed on their archival materials to their associates or successors or to libraries, museums, and educational institutions. A thorough search of the various organizations' files and archival sources is often highly cost effective.

4. HISTORICAL BUILDINGS

Historical buildings deemed to be of importance to the history, architecture, or culture of an area or community, and so designated by appropriate public agency, require special consideration.

Evaluation of residual structural reliability requires special skills, and, in some cases, the standards of structural modification may be relaxed in order to preserve the original character of the building at an acceptable cost.

5. IDENTIFICATION OF DESIRED CHANGES IN THE BUILDING

The space modifications may be relatively minor, with addition or removal of only a few partitions, or it may be major with total "gutting" of the interior and "re-stuffing" it with new floors, partitions, stairwells, elevator shafts, etc. In some cases, the exterior walls will undergo only minor modifications, perhaps cosmetic only; in other cases, strengthening of exterior walls may be necessary. Floors and roofs can often 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.

Assessment of fire safety may lead to a variety of required modifications: installation of fire detection and suppression systems.

6 Bresler

system, addition of fire proofing, fire separation walls, fire doors and other exits, and, in some cases, addition of stairs or elevators. Trade-offs between alternative solutions would determine the final architectural space modifications.

Assessment of the need for structural modification is based primarily on evaluation of the performance of the building exposed to a variety of possible loading and environment conditions such as excessive live load, snow, wind, earthquake, fire, etc. Given a specific building, proposed occupancy and space changes, and given loading and environment requirements, the force and deformation response of structural elements can be calculated and compared to the estimated capacity. For satisfactory performance, the resistance capacity should exceed the response demand of an acceptable margin. This margin is a measure of the level of performance. The process is illustrated schematically in Figure 1.

Modeling the structural system and the various loading conditions, as well as calculating the response demand and capacity, follow the generally accepted procedures of structural analysis and design. As a minimum requirement, this process requires a review of the mandated level of structural adequacy established by a governmental agency having jurisdiction; the review must be based on present state-of-the-art and existing conditions in the structure. Thus, given the proposed occupancy and architectural space modification, structural sufficiency (for superstructure as well as the foundation) must be verified for gravity loads, combined gravity and lateral loads, as well as concentrated loads, impact, construction loads during modification, and special service conditions which may be appropriate for the case under consideration.

It is interesting to compare design live loads for different types of buildings recommended by different cities in 1900 with typical live loads specified in contemporary codes (see Table 2). In most cases, contemporary new buildings are designed for smaller live load requirements compared to those used in the design of old buildings. Also of interest are allowable stresses for concrete and masonry, as shown in Table 3.

Allowable stresses for concrete varied during different periods of time. Particularly significant were the variations in allowable stresses for shear in reinforced concrete. The progressive changes in these values for concretes of different strength are shown in Table 4. Except for a brief period of time, 1940-1956, when higher values of shear stresses were permitted, the general trend for allowable stresses in concrete is consistent. It should be noted, however, that prior to 1940, concretes having design compressive strengths in excess of 3000 psi were rarely used.

The strength of an unreinforced brick wall in compression and shear depends primarily on mortar. In the old codes, no

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values of shear allowable were given, but distinction was made between brick walls made with portland cement mortar and those made with lime mortar. It is interesting to note that there is relatively little change in allowable values over the years, although newer types of masonry - both concrete block and reinforced brick - have been developed for which increases in allowable stress are significant. This suggests that, provided there is no aging degradation in the construction materials, no reduction in allowable stresses for the type of construction considered, no requirements for the building under the new occupancy to withstand higher loads than those originally designed for, and no removal of gravity bearing structural elements in the spatial remodeling of the building, the structural modification for support of gravity loads may be minor.

6. REVIEW OF CODE REQUIREMENTS

Codes are continually revised to keep pace with changes in construction technology reflecting innovation in materials, structural systems and quality control methods, as well as new knowledge of structural behavior acquired through analysis, testing, and experience. Code revisions also include deletion of sections dealing with materials and methods of construction no longer in use.

While the code revisions play an important role in the advance of technology of new construction, the current codes often complicate the process of rehabilitating old buildings. The new codes are poor instruments for evaluating adequacy of existing buildings: non-compliance with current code does not mean poor performance of an old building. In many cases, old types of construction are just not covered in a current code. Upgrading an old building to comply with current standards for new construction is generally extremely difficult and costly as full upgrading often requires extensive demolition and reconstruction, and its economic impact can result in the abandonment of a rehabilitation project.

It is generally advisable during the preliminary stages of design to consult with local building officials to provide them with a chance for early input into rehabilitation process. At this time, the building official will have an opportunity to discuss with the designers what level of structural rehabilitation would be acceptable for the particular building. The official might also review preliminary schemes for reinforcement of different elements and may indicate a tentative approval of these methods.

Current code provisions on rehabilitation are based on a "passive" approach; i.e., assuming that old buildings are essentially tolerated until the owner chooses to remodel or change the use of the building. If the extent of proposed

8 Bresler

changes exceeds certain threshold levels, then the portion of the building affected, if not the entire building, is required to be upgraded.

Typical code requirements for alterations, additions, and repairs of existing buildings based on extent and nature of alterations are summarized in Table 5.

These requirements are based on the following criteria:

1. Extent of Alteration: This is defined either in terms of the value of alterations expressed as % of the value of the existing building, or in terms of space involved in alteration expressed as % of the total space in the existing building.
2. Nature of Alteration: Generally, the alternations are subdivided into structural and non-structural alteration, the latter including all architectural, mechanical and electrical systems.
3. Increment of Alteration: Generally, any single or multiple alterations within a 12-month period are considered as the increment of alteration defining the extent of alteration. In some cases, all alterations cumulative since the building was built are included in defining the extent of alteration.
4. Change of Occupancy: Categories of buildings which fall in specific occupancy groups are defined in the code. Typical occupancy categories include: public assembly, schools, hospitals, commerical, industrial, hotels, and apartment houses and dwellings. Change in use or occupancy is accompanied by change in the number of occupants, change in the floor loading, change in the building contents, and, generally, changes related to life, fire, and seismic hazard risk.

The code requirement related to change in use usually states that when a more hazardous use for a given building is proposed, the building should comply with all the requirements of the current code - particularly with respect to life, fire, and seismic hazards. When the new use is less hazardous, the entire building, subject to approval by the building official, need not comply with all the code requirements for the particular occupancy group. Some codes provide that when the change in occupancy increasing the hazard involves less than 30% of building space, only that space, not the entire building, need conform to the occupancy requirement.

7. ARCHITECTURAL AND STRUCTURAL MODIFICATION

Design professionals view rehabilitation projects differently according to the responsibility that each must assume. Architectural modifications focus on the reallocation of space for different needs, on the traffic flow within the building, on services, communications, and general environment in the building. Thus, the architect will be concerned with relocating walls and partitions, entrances and exits, stairs and elevators, toilets and other plumbing, lighting, exterior cladding and interior finishes, telephones and electric wiring, heating and air conditioning, fire detection, alarms, and suppression systems, and, in general, creating a pleasant environment in the space for the new use of space.

The structural engineer is concerned with the adequacy of the structure to carry gravity loads, lateral loads (wind, seismic), and the response of the structural system to such environmental conditions as changes in temperature and humidity. In rehabilitation of existing buildings, the structural engineer will focus on condition and strength of structural materials in the buildings, adequacy of structural floor and roof framing, vertical framing and walls, and adequacy of foundation system. Strengthening may be required as a result of removal of some walls or portions of floors necessitated by architectural space modifications, as a result of material deterioration, or as a result of an increase in loading due to a change in the use of the building.

One of the deficiencies in the design of old buildings is lack of provisions for lateral loadings - wind or seismic. The problem of seismic rehabilitation is a special problem and will be briefly addressed in a subsequent section. As for effects of wind on old buildings, it is important to ensure that the modifications in the old structure do not reduce its capability to resist wind. Anchoring the roof to the remaining structure, bracing of the roof, and the effectiveness of the remaining (or new) floors to act as horizontal diaphragms in distributing the lateral load to the walls or frames must be ensured.

The type of structural modification in a given building depends on the deficiencies discovered in the existing system. Generally, the modifications can be subdivided into two categories:

- a. maintaining the original structural system and strengthening critical members or connections, and,
- b. modifying the original structure by replacement of floor systems, vertical framing, and/or addition of structural elements to the vertical load carrying system or lateral bracing system.

10 Bresler

Prior to the development of a structural modification scheme, an evaluation of the condition of existing structures and structural materials must be carried out. This requires testing in situ, or removal of specimens for testing in the laboratory. Assuming excessive degradation of the structure or of the materials may lead to costly and unnecessary modifications, and, in some cases, to unconservative modification schemes.

8. STRENGTHENING OF OLD BUILDINGS FOR EARTHQUAKE RESISTANCE

Many old buildings were designed prior to the introduction of adequate earthquake regulations and used types of construction which subsequently proved vulnerable to earthquake damage. Other old buildings may be vulnerable to damage because they deteriorated due to a variety of conditions, such as previous earthquake damage, fire damage, foundation settlement, or alterations. Most of these are old low-rise buildings, including apartment buildings, commercial, industrial, or public buildings constructed without adequate provision for ductile response or without adequate horizontal bracing or membrane systems tying together the vertical elements of the building.

Where seismic exposure risk is significant, alteration of existing buildings to conform to current code standards of earthquake resistant design is often not feasible. There is a rapidly growing concern, especially in California, for the seismic safety of existing buildings. This concern has sparked the trend towards a more "active" approach to rehabilitation of older buildings. For example, in 1971, the Santa Rosa City Council adopted Resolution 9820 which provides procedures for the systematic survey and reconstruction of existing buildings. The resolution establishes a priority system for building review based on occupancy (theaters and hotels before markets, and offices before apartments and warehouses). The city is required to perform a preliminary review of the buildings in their order of priority, to identify any existing fire or existing seismic hazards, to determine whether corrective work is needed and to furnish the property owner with a report identifying any deficiencies found. If any rehabilitation is necessary, the resolution requires the owner to submit detailed plans, prepared by a licensed engineer, to the city to obtain a building permit for the required work. The work must be completed within one year after receipt of the report from the city. To enforce the resolution, the city is empowered to order vacation and demolition of the building if the owner fails to perform the necessary rehabilitative work. Recognizing the impact such a program would have on their community and attempting to make the reconstruction economically feasible, the review is limited only to those buildings constructed prior to 1958, and requires compliance with the provisions of the 1955 Uniform Building Code.

Other California cities have also undertaken similar rehabilitation programs designed to abate safety hazards in

existing buildings. Long Beach was the first city to establish an earthquake safety program for the rehabilitation of pre-1934 buildings. San Diego and Sacramento have incorporated rehabilitation requirements into their redevelopment programs in an effort to preserve historic buildings. Los Angeles has passed a new ordinance in 1981 which has developed a comprehensive approach for abatement of seismic hazards of existing buildings that is somewhat similar to the ATC-3 recommendation for seismic hazard abatement in existing buildings. San Francisco has a parapet ordinance and has also taken the first steps towards development of an ordinance for abatement of seismic hazards in older buildings.

While the programs indicated above may differ in scope and detail, they generally have several characteristics in common such as an established buildings priority system, a defined review process, a requirement to perform any required rehabilitation work within a specified time period, penalties for failure to comply with the requirements of the ordinance, and a set of criteria usually less stringent than those currently required for new construction to which the existing building must comply. There is no doubt that the type of program embodied by such an ordinance is the future trend in standards of rehabilitation.

9. DRAWINGS, SPECIFICATIONS AND CONSTRUCTION

Following development of the structural modification scheme and review and approval of this preliminary design by the owner and the building official, the design team, including architects and engineers, will begin development of working drawings and specifications for the rehabilitation project. These documents will be used in the field by the contractor to complete the required work, and, depending upon the size and the nature of the project, may provide for demolition, site development, civil, architectural, structural, mechanical and electrical work. At this time, the structural engineer must refine his analyses and designs for the structural modifications which involve selection of all new structural elements, their sizing, reinforcement, and interface connections to the existing structure.

In rehabilitation and renovation work, the coordination between various disciplines is a particularly difficult problem. The architectural, mechanical, and electrical requirements may necessitate penetrations of structural members, and the structural engineer must ensure that these are accommodated without degrading the structural integrity of the building. Also, addition of new strengthening elements must not interfere with the intended functional use of the building and must not conflict with the aesthetic requirements of the environment. The general spatial restrictions normally characteristic of rehabilitation work also compound the coordination problem.

In preparing specifications and contracts for rehabilitation projects, provisions should be made for changes caused by

12 Bresler

unanticipated conditions in the building discovered during the construction. When encountered, these conditions may require significant alteration of proposed structural modifications in the existing structure. Such changes, if not properly provided for in the specifications and contract documents, can cause significant delays and added costs.

While many rehabilitation or renovation projects go through the normal bidding process in selecting a contractor, it is often desirable to negotiate a contract with a selected qualified contractor who then becomes a member of the design team in the early stages of the project.

On large rehabilitation projects, it is particularly important to provide adequate inspection, continuous or intermittent, at the construction site. Employment of a reliable testing laboratory to monitor the quality of materials and methods of construction on a rehabilitation job is even more important than it is on new construction.

10. CONCLUDING REMARKS

Structural evaluation of existing buildings differs in several respects from design of new buildings. On the one hand, the engineer can determine the "as-built" conditions with greater certitude than that of the assumptions usually made with regard to designed buildings yet to be constructed. On the other hand, the existing buildings have locked within their walls (as well as within their frames and their floors) mysteries of the "as-built" conditions which are difficult to determine short of destroying the building.

The engineer has a greater number of choices in the design of new buildings (yet to be constructed) than in the design of modifications for existing buildings; this places constraints on the type of structural rehabilitation although many different solutions are still available to engineers experienced in repair and strengthening of existing buildings.

At the turn of the century, the engineer played a more important role in the construction of a building. Moreover, the skilled construction workers knew how to follow instructions of the engineer without detailed drawings. Let me illustrate this by the following quotation from a 1909 specification:

"All concrete must be mixed by a batch machine mixer as approved by the architect. The aggregate and water for each batch must be precisely and regularly proportioned as specified and first mixed dry until the mixture is uniform and then the water must be added while the mixing is contained so as to produce a mixture of uniform color and consistency. Only laborers of intelligence

and ability to understand the directions given them are to be employed to mix and place materials in the mixer."

Today, such language is not considered acceptable by the construction industry and design professionals.

The problem of rehabilitating old buildings poses a fascinating challenge to the engineering profession. It demands of them powers of deduction no less than those attributed to Sherlock Holmes! It demands of them knowledge of the recent history of the builders' art, the art of old designers and constructors alike. It demands of them knowledge of all the new scientific tools, including pachometers, gamma ray scanning, sonoscopes, pulse echo devices, and even the maligned computer, all of which can help penetrate the mysteries locked up in the concrete and steel skeleton of a building. Last, but not least, it demands of them the skill and experience of an imaginative and experienced contemporary structural designer who would be able to blend the new materials and new structural concepts with the old building. In blending these, he must be sensitive to the aesthetic demand and the respect which the old buildings deserve, but he must not give up his primary responsibility for making the old buildings perform their new roles safely and without malfunction.

Some would say that all this is too much to ask from a structural engineer. I disagree: it is not too much to ask, even if getting all of this from any one man may be well nigh impossible. But, it certainly is not too much to ask from a team of people involved with renovation of old buildings. We can ask no less! And we can take a measure of pride that, on the whole, the structural engineering profession plays its role in the process of renovation with sensitivity and competence.

14 Bresler

TABLE 1 - ESSENTIAL STEPS IN ADAPTATION OF OLD BUILDINGS1. PRELIMINARY SITE VISIT

Make a preliminary visual inspection, review proposed new use.

2. DOCUMENTATION OF ORIGINAL DESIGN AND CONSTRUCTION

Locate design or as-built drawings, specifications, calculations, and interim modifications, if available. If not available, prepare documentation on "as is" condition using field survey.

3. DESIRED CHANGES IN THE BUILDING

Identify required changes for adaptation to new use.

4. CODE REQUIREMENTS

Based on new occupancy and space renovation, identify code requirements which must be met to comply with structural and non-structural requirements applicable to building alterations.

5. ARCHITECTURAL AND STRUCTURAL MODIFICATIONS - COST ESTIMATE

Develop architectural and structural modification scheme and make preliminary assessment of feasibility and cost (see Step 8). If these are acceptable, then proceed with implementation of the proposed adaptation. Sometimes, several alternatives have to be studied before one is selected.

6. STRUCTURAL ADEQUACY

Identify "as is" condition of materials and details of construction. Verify adequacy of the proposed modification scheme and review feasibility and preliminary cost estimate.

7. FINAL DESIGN AND DOCUMENTATION

Complete analysis and design of structural modification; interface with other non-structural modifications; prepare drawings, specifications, contract documents.

8. CONSTRUCTION

Select contractor, develop quality control program (inspection and testing), accommodate possible need for design changes. This step is often incorporated in Step 5, resulting in substantial saving in time and in cost. Proceed with construction ensuring close communication between field and office to accommodate possible need for design changes caused by unexpected conditions discovered in the process of construction.

Adaptation of Old Buildings 15

TABLE 2 - LIVE LOADS FOR FLOORS IN DIFFERENT CATEGORIES OF BUILDINGS

POUNDS PER SQUARE FOOT

Occupancy	CODES				
	New York 1900	Chicago 1898	Philadelphia 1899	Boston 1899	UBC 1973
Apartments, Hotels	60	40	70	50	40
Office Buildings	75	100	100	100	50 ⁽¹⁾
Schools	75	--	--	80	40 ⁽²⁾
Public Assembly	90	100	120	150	50-125 ⁽³⁾
Light Manufacturing	120	100	120	--	75 ⁽¹⁾
Heavy Manufacturing	150	--	150	250	125 ⁽⁴⁾

1) Concentrated load 2000 lbs.

2) Concentrated load 1000 lbs.

3) First number-fixed seating areas;
last number-stage areas and
enclosed platforms.

4) Concentrated load 3000 lbs.

TABLE 3 - ALLOWABLE UNIT STRESSES FOR

DIFFERENT MATERIALS AND CONDITIONS

POUNDS PER SQUARE INCH

Materials	CODES				
	New York 1900	Chicago 1898	Philadelphia 1899	Boston 1899	UBC 1973
<u>CONCRETE</u>					
Compression	230	55	210	--	500 ⁽¹⁾
Bending	--	--	--	--	1,030 ⁽²⁾
Shear	--	--	--	--	--
	(See Table 3)				
<u>BRICK</u>					
Compression	110-250	--	110-210	--	100-750
Bending	30	--	--	--	6- 40
Shear	--	--	--	--	6- 40

1) $f_c = 0.2f'_c = 500$ psi2) $f_b = 0.45f'_c = 1,030$ psi

16 Bresler

TABLE 4 - ALLOWABLE UNIT SHEAR STRESS (V/bd) psi FOR
REINFORCED CONCRETE BEAMS - VARYING WITH CODE CHANGES

CODE	DATE	Concrete Design Strength f'_c , 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 adjusted for $j = 7/8$

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

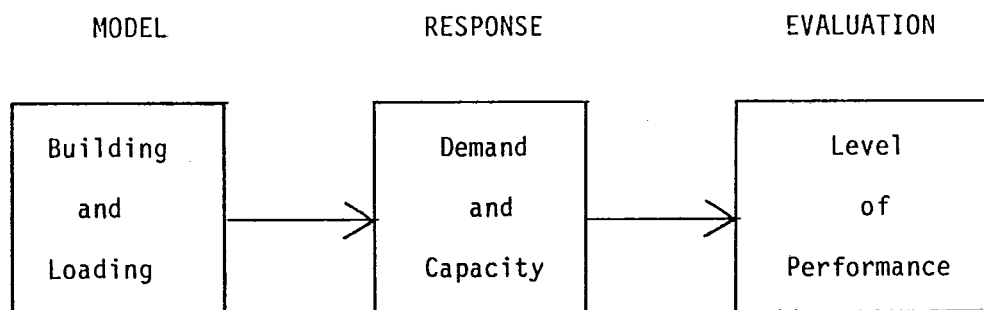
** For values of shear stress greater than the lower limit --all shear
must be taken by web reinforcement

*** Appendix B - Alternate Design Method is applicable for designs
using service loads and allowable unit values of working stress.

TABLE 5 - TYPICAL CODE REQUIREMENTS FOR
ALTERATIONS OF EXISTING BUILDINGS

Extent of Alteration (Value or Space)	Nature of Alteration	
	Structural	Non-Structural
$V > 50\%$	Entire building must conform.	Entire building must conform.
$50\% > V > 25\%$	New construction must conform.	New construction must conform.
	Entire building need not conform.*	Entire building need not conform.
$25\% > V$	New construction must conform	May be made same as original construction
	Minor changes, subject to approval by Building Official, may be made same as original construction.	

* In some cases, all floors below new construction should conform.



1. Fig. 1--Schematic diagram of structural evaluation

SP-85-2

Nondestructive Evaluation in Rehabilitation and Preservation of Concrete and Masonry Materials

By James R. Clifton

Synopsis: This paper describes nondestructive evaluation (NDE) methods that can be used in assessing the condition of concrete and masonry materials and components in structures being rehabilitated or preserved. Metal reinforcement is also included. The appropriate use of NDE methods is discussed and a recommended approach to selecting NDE methods for specific situations is given.

NDE methods are briefly described based on their principle of operation, information they provide, applications, expertise required to use them, and advantages and limitation to their use.

Keywords: concretes; evaluation; inspection; masonry; nondestructive tests; reinforcing steels; renovating.

20 Clifton

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INTRODUCTION

An important phase in the rehabilitation or preservation of concrete and masonry structures is making decisions on the need to repair, reinforce, or replace concrete and masonry materials and building components. Information on the condition of the concrete and masonry materials, including any internal metal reinforcement, is usually required before such decisions are made. The traditional process of condition assessment includes making a visual inspection of the structure, characterizing the important properties of a material using cores and other samples, and performing structural analyses. For condition assessment purposes, the use of nondestructive evaluation (NDE) methods has gained increased acceptance as a rapid and reliable approach. The use of NDE for the assessment of the condition of concrete and masonry materials in situ is discussed in this paper.

Some of the methods described in this report cause some cosmetic damage to the material being inspected. The service life of the material, however, is not affected and the slight damage can be readily repaired, if desired. Such methods, therefore, are usually considered to be nondestructive.

APPROPRIATE USE OF NDE

NDE does not entirely eliminate the need for taking samples and analyzing or mechanically testing them, but can substantially reduce the amount of such testing. For example, cores are often taken if concrete appears to be substantially below the required quality or strength. Taking and testing numerous cores from a concrete structure is undesirable for several reasons including the time consumed, the expense, and the damage to the structure. NDE can be used to identify those areas that should be cored, thereby reducing the number of cores needed to quantitatively evaluate the concrete. Further, by properly using and interpreting the results of NDE techniques such as ultrasonic pulse velocity, rebound hammer, or penetration probe, the uniformity or quality of a large area of concrete can be rapidly surveyed.

Another application of NDE in rehabilitation and preservation is the identification of internal construction

details, such as location, size, shape, and corrosion condition of reinforcing bars in concrete and masonry walls. The condition and presence of materials and service systems (electrical wiring, pipes, etc.) in cavity walls and floors can be determined using a fiberscope. Several NDE techniques are addressed in this paper which can be used to detect subsurface voids and delaminations in concrete and masonry walls, facades, and cornices.

With the possible exception of pullout methods, NDE methods do not directly measure strength values. Rather, empirical relationships must be developed between strength measurements and NDE test data taken on the same concrete. The recommended method for developing these relationships is to test concrete cores taken from the concrete in question. If test specimens cannot be removed, a reasonable alternative is to make test concrete of the same composition and porosity as the concrete in question and test it to develop the necessary relationships. The relationships developed based on reconstituted concrete, however, should be used with caution because factors other than compositional variables influence the strength development of concrete, such as consolidating practice and curing conditions.

SELECTION OF NDE METHODS

The selection of NDE methods for assessing the condition of concrete, masonry, and metal reinforcement should be based on considerations of the important physicochemical and engineering properties which need to be measured. For example, the information needed on reinforced concrete may include its strength, quality, uniformity, stiffness, density, and location and conditions of steel reinforcement. NDE methods which can be used for obtaining this information are given in Table 1. Similarly, NDE methods which can be used for inspecting masonry are listed in Table 2. The Recommended Test Methods in Tables 1 and 2 are commonly used and their limitations are considered to be adequately understood, while the reliability and feasibility of Possible Test Methods are still being assessed.

DESCRIPTION OF NDE METHODS

The operation, principles, main applications of the NDE methods given in Tables 1 and 2 are outlined in Table 3. The User Expertise comments address relative competence levels and the brief comments under Advantages and Limitations are intended to indicate the feasibility of using the individual NDE methods. The overall purpose of Table 3 is to provide a brief description.

22 Clifton

of the NDE methods which could be used in assessing the condition of concrete, masonry, and metal reinforcement. More detailed information on these and other NDE methods used for the inspection of building materials can be found in reference numbers 1-6.

ASTM STANDARDS FOR NDE METHODS

The American Society for Testing and Materials (ASTM), has issued standards for NDE methods. Some of the standards pertaining to the use of NDE for inspecting concrete and masonry are given in Table 4. These standards provide information on the proper use of NDE equipment or interpretation of test results.

SUMMARY

NDE methods are available which can be used in assessing the in situ condition of concrete, masonry, and their reinforcing metals. While NDE does not eliminate the need for taking samples and testing them in the laboratory, it can substantially reduce the amount of laboratory testing.

In the NDE inspection of concrete and masonry materials and components, their important properties or performance requirements should first be identified and then appropriate NDE methods selected. In this report many of the important properties of concrete and masonry are given and appropriate NDE methods recommended. NDE methods are described based on their principle of operation, information they provide, their main applications, expertise required to use them, and advantages and limitations to their use.

ACKNOWLEDGMENTS

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24 Clifton

Table 1. NDE Methods for Inspecting of Concrete Materials

Concrete Properties	Recommended NDE Methods	Possible NDE Methods
Strength	(1) Penetration Probe (2) Rebound Hammer (3) Pullout Methods	
General Quality and Uniformity	(1) Penetration Probe (2) Rebound Hammer (3) Ultrasonic Pulse Velocity (4) Gamma Radiography	Ultrasonic Pulse Echo
Thickness		(1) Radar (2) Gamma Radiography (3) Ultrasonic Pulse Echo
Stiffness	Ultrasonic Pulse Velocity	Proof Loading (Load-deflection)
Density	(1) Gamma Radiography (2) Ultrasonic Pulse Velocity	Neutron Density Gage
Rebar Size and Location	(1) Covermeter (2) Gamma Radiography	(1) X-ray Radiography (2) Ultrasonic Pulse Echo (3) Radar
Corrosion State of Reinforcing Steel	Electrical Potential Measurement	
Presence of Subsurface Voids	(1) Acoustic Impact (2) Gamma Radiography (3) Ultrasonic Pulse Velocity	(1) Thermal Inspection (2) X-ray Radiography (3) Ultrasonic Pulse Echo
Structural Integrity of Concrete Structures	Proof Loading (Load-deflection)	Proof Testing Using Acoustic Emission

Table 2. NDE Methods for Inspecting Masonry Materials

Properties	Recommended NDE Methods	Possible NDE Methods
Integrity of Masonry	(1) Acoustic Impact (2) Gamma Radiography (3) Probe Holes with Fiberscope (4) Ultrasonic Pulse Velocity	Ultrasonic Pulse Echo
Thickness of Masonry	Probe Holes	(1) Eddy Current (2) Ultrasonic Pulse Echo
Reinforcing Steel (Location and Size)	(1) Covermeter (2) Gamma Radiography	(1) Ultrasonic Pulse Echo (2) X-ray Radiography
Presence of Inner Grout	(1) Probe Holes (2) Gamma Radiography (3) Acoustic Impact (4) Ultrasonic Pulse Velocity	Ultrasonic Pulse Echo
Moisture in Masonry Walls	Moisture Meter-Electrical Resistance	Moisture Meter-Neutron
Presence of Delaminations	(1) Acoustic Impact (2) Gamma Radiography (3) Ultrasonic Pulse Velocity	(1) Ultrasonic Pulse Echo (2) Thermal Inspection (Thermography) (3) X-ray Radiography

Table 3. Survey of NDE Methods for Inspecting Concrete, Masonry, and Reinforcing Metals

Method	Principle	Main Applications	User Expertise	Advantages	Limitations
1. Acoustic Emission	During crack growth or plastic deformation, the rapid release of strain energy produces acoustic (sound) waves that can be detected by sensors attached to the surface of a test object.	Continuous monitoring of structure during service life to detect impending failure; monitoring performance of structure during proof testing.	Extensive knowledge required to plan test and to interpret results.	Monitors response of as-built structure to applied load; capable of detecting onset of failure; capable of locating source of possible failure.	Requires means of loading structure; complex electronic equipment is required.
2. Acoustic Impact (Hammer Test)	Surface of object is struck with a hammer. The frequency and damping characteristics of the "ringing" can give an indication of the presence of defects.	Detect delamination or disbands in composite systems; detect voids and cracks in materials, e.g., hammer technique detects defective masonry units; "chain drag" method to detect delaminations in concrete pavements.	Low level of expertise required to use, but experience needed for interpreting results.	Portable; easy to perform test; electronic device not needed for qualitative results.	Geometry and mass of test object influences results; poor discrimination; reference standards required for electronic testing.
3. Covermeter	Presence of steel affects the magnetic field of a probe. Closer probe is to steel, the greater the effect. Principle of operation is similar to eddy current method.	Determination of presence, location and depth of rebars in concrete and masonry units.	Moderate, easy to operate, need some training to interpret results.	Portable equipment; good results if concrete is lightly reinforced.	Difficult to interpret results if concrete is heavily reinforced or if wire mesh is present.
4. Electrical Potential Measurements	Electrical potential of steel reinforcement measured. Potential indicates probability of corrosion.	Determining condition of steel rebars in concrete and masonry.	Moderate. User must be able to recognize problems.	Portable equipment; field measurements readily made; appears to give reliable information.	Does not provide information on rate of corrosion. Requires access to reinforcing bars to make electrical contact.
5. Fiberscope (Endoscope)	Bundle of flexible, optical fibers with lens and illuminating systems is inserted into small bore holes thus enabling view of interior of cavities.	Check condition of materials in cavity, such as pipes and electrical wiring in cavity walls; check for unfilled cores in reinforced masonry construction; check for voids along grouted stressed tendons.	Low.	Direct visual inspection of otherwise inaccessible parts is possible.	Probe holes usually must be drilled; probe holes must connect to a cavity.
6. Gamma Radiography	Gamma radiation attenuates when passing through a building component. Extent of attenuation controlled by density and thickness of the materials of the building component. Photographic film record usually made, which is analyzed.	Locating internal cracks, voids and variations in density of materials. Locating internal parts in a building component, e.g., reinforcing steel in concrete.	Must be operated by trained and licensed personnel.	Portable and relatively inexpensive compared to X-ray radiography; internal defects can be detected; applicable to a variety of materials.	Radiation intensity cannot be adjusted; long exposure times may be required; dangerous radiation; two surfaces of component must be accessible.
7. Moisture Meter-Electrical Resistance	Electrical resistance between two probes inserted into test component is measured. The resistance decreases with increased moisture contents.	Measurement of moisture contents of timber, roofing materials and soils. Can be used to map moisture migration patterns in masonry walls.	Low.	Equipment is inexpensive, simple to operate and many measurements can be rapidly made.	Not reliable at high moisture contents; needs to be calibrated; precise results are not usually obtained.

Table 3. Survey of NDE Methods for Inspecting Concrete, Masonry, and Reinforcing Metals (continued)

Method	Principle	Main Applications	User Expertise	Advantages	Limitations
8. Moisture Meter-Neutron	Fast neutrons are slowed by interactions with hydrogen atoms. Back-scattered slowed neutrons are measured, the number of which are proportional to the amount of hydrogen atoms present in a material.	Moisture content measurements of soil and roofing materials. Can be used to map moisture migration patterns in masonry walls.	Must be operated by trained and licensed personnel.	Portable; moisture measurements can rapidly be made on in-service materials.	Only measures moisture content of surface layer (50 mm); dangerous radiation; hydrogen atoms of building materials are measured in addition to those of water.
9. Nuclear Density Meter	Gamma rays are used to measure mass density. The energy loss of the emitted gamma rays is proportional to the mass density of the material through which the rays pass.	Measurement of density of soils. Could be used for estimating density of concrete.	Must be used by trained and licensed personnel.	Portable; density measurements can be made without disturbing the material being tested.	Calibration necessary; dangerous radiation.
10. Penetration Probe	Probe fired into concrete and length of nonpenetration portion is measured. Surface and subsurface hardness measured.	Estimations of compressive strength, uniformity and quality of concrete. Could be used for estimating the same properties of mortars.	Low, can be operated by field personnel.	Equipment is simple and durable; good for determining quality of concrete.	Slightly damages small area; does not give precise prediction of strength.
11. Proof Loading	Structure or system is subjected to loads and response is measured.	Determining safe capacity and integrity of structures. Leak testing of pressure vessels and plumbing.	Depends on nature of tests; can be high.	Entire structure can be tested in its "as-built" condition.	Can be very costly; instrumentation required to measure response; careful planning required; can damage structure.
12. Pullout Devices	Measure the force required to pull out a steel rod with enlarged head cast in concrete. Also, pullout device can be inserted in hole drilled in hardened concrete. Pullout forces produce tensile and shear stresses in concrete.	Estimation of compressive and tensile strengths of concrete.	Low, can be used by field testers and concrete inspectors.	Only NDE method which directly measures in-place strength of concrete; appears to give good prediction of concrete strength.	Pullout devices must be inserted during construction, or inserted in hole drilled in hardened concrete. Cone of concrete may be pulled out, necessitating minor repairs.
13. Radar	Radio frequency waves from a radar transmitter are directed into a dielectric material. The waves propagate through material until a boundary is intercepted, then part of the incident energy is reflected. The reflected (echo) wave is picked-up by a receiver. A boundary exists at discontinuities or at interfaces between materials of different dielectric properties.	Detection of interfaces, delaminations and voids in concrete and measurement of thickness of concrete pavements.	High, requires an understanding of wave propagation in materials.	Large area of concrete can be rapidly surveyed; internal construction details and defects can be identified.	Further research required to establish a technical basis for interpreting test data. Interpretation of data may require use of computers.

28 Clifton

Table 3. Survey of NDE Methods for Inspecting Concrete, Masonry, and Reinforcing Metals (continued)

Method	Principle	Main Applications	User Expertise	Advantages	Limitations
14. Rebound Hammer	Spring driven mass strikes surface of concrete and rebound distance is given in R-values. Surface hardness is measured.	Estimation of compressive strength, uniformity and quality of concrete.	Low, can be readily operated by field personnel.	Inexpensive; large amount of data can be quickly obtained; good for determining uniformity of concrete.	Results affected by condition of concrete surface; does not give precise prediction of strength.
15. Thermal	Heat sensing devices are used to detect irregular temperature distributions due to presence of flaws or inhomogeneities in a material or component that has different impedances to heat flow. Contours of equal temperature (thermography) or temperatures (thermometry) are measured over the test surface with contact or non-contact detection devices. A common detection device is an infrared scanning camera.	Detection of heat loss through walls and roofs; detection of moisture in roofs; detection of delamination in composite materials, including concrete and masonry walls.	Moderate to extensive depending on nature of test.	Portable; permanent record can be made; testing can be done without direct access to surface and large areas can be rapidly inspected using infrared cameras.	Equipment costly; reference standards needed; means of producing thermal gradient in test component or material may be required.
16. Ultrasonic Pulse Echo	Pulsed compressional waves are induced in materials and those reflected back are detected. Both the transmitting and receiving transducers usually are contained in the same probe.	Inspecting metals for internal discontinuities. Some work has been performed on the use of the pulse echo method to inspect concrete for delaminations and cracks.	High level of expertise; require to interpret results.	Portable; internal discontinuities can be located and their sizes estimated.	Good coupling between transducer and test substrate critical; interpretation of results can be difficult; calibrating standards required.
17. Ultrasonic Pulse Velocity	Based on measuring the transit time of an induced pulsed compressional wave propagating through a material.	Estimation of the quality and uniformity of concrete.	Low level required to make measurements. Expertise needed to interpret results.	Excellent for determining the quality and uniformity of concrete. Can rapidly survey large areas and thick members.	Does not provide estimate of strength; skill required in analysis of results, moisture variations and presence of metal reinforcement can affect results.
18. X-ray Radiography	Similar to gamma radiography, except X-rays are used.	To identify hidden construction features in wooden structures. Could be used for inspecting thin concrete components.	Should be operated by trained personnel because of radiation.	Intensity of radiation can be varied.	Dangerous radiation; portable units have low intensities and field applications limited to wooden and thin components; opposite surface of component must be accessible.

Table 4. ASTM Standards for Some NDE Methods for Concrete and Masonry Materials

NDE Method	ASTM Standards and Designation
Rebound Hammer	Test for Rebound Number of Hardened Concrete: C805
Penetration Probe	Test for Penetration Resistance of Hammer Concrete: C803
Ultrasonic Pulse Velocity	Test for Pulse Velocity through Concrete: C597
Nuclear Moisture Meter*	Standard Test Method for Moisture Content of Soil and Soil-aggregate in place by Nuclear Methods (shallow depth): D3017
Cast-in-Place	Test Method for Pullout Strength of Hardened Concrete: C900
Radiography+	(1) Controlling Quality of Radiographic Testing: E142 (2) Recommended Practice for Radiographic Testing: E94
Ultrasonic Pulse-Echo+	Practice For Ultrasonic Pulse-Echo Straight-beam Testing by the Contact Method: E114

*While the standard does not specifically cover the testing of concrete and masonry, the nuclear moisture meter can be used to detect moisture in them.

+Standard covers test procedures and do not specifically address concrete.

SP-85-3

Inspection, Analysis, and Restoration of MIT Kresge Auditorium

By Edward Cohen, Norval Dobbs, and William Combs

Synopsis: This paper describes the engineering procedures involved in the renovation of Kresge Auditorium which is located at the Massachusetts Institute of Technology in Boston. Described are inspection procedures used to determine any structural deterioration which may have occurred since the building's inception. These procedures included visual inspection to determine cracking and other aspects which would possibly reduce the structure strength, pulse-echo test and chemical and petrographic tests of the structure's concrete as well as continuous monitoring of structure motions during remedial construction. To assist in this remedial work, a series of finite element analyses were performed to determine stress levels within the structure. These analyses considered initial construction stresses as well as increased stresses due to structure deterioration. Based on the above, a series of structural modifications were developed and implemented in the structure restoration.

Keywords: auditoriums; cracking (fracturing); deterioration; finite element method; inspection; nondestructive tests; reinforced concrete; renovating; shells (structural forms); stress analysis.

32 Cohen, Dobbs, and Combs

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Norval Dobbs, senior associate Ammann & Whitney, was past recipient of the Laskowitz Gold Medal Award of the New York Academy of Sciences.

William Combs, member of the Department of Physical Plant, Massachusetts Institute of Technology, is responsible for a large portion of on campus construction and facilities coordination.

INTRODUCTION

General

In 1950, Eero Saarinen and Associates of Bloomfield Hills, Michigan, were commissioned by the Massachusetts Institute of Technology to design an auditorium "to seat 1,200 persons together with a small intimate theatre to accommodate a further 200, the site to be in the heart of the great campus, which has a superb location on the banks of the Charles River, overlooking the centre of Boston". Ammann and Whitney, Consulting Engineers of New York City were retained as structural engineers. Construction of the auditorium, which was named "Kresge Auditorium", was completed in 1954 and received international acclaim.

The roofing system, installed initially and replaced repeatedly, did not prevent the entrance of moisture as evidenced by seepage at the abutments and was the primary cause of their deterioration. Consequently, as part of their overall responsibility for the maintenance of existing facilities and construction of new facilities, the Department of Physical Plants (DPP) of the Institute undertook in 1979 a major maintenance effort for the Auditorium. This effort included the replacement of the existing roof and the repair of all structural degradations.

Description of Structure

Kresge Auditorium consists of a reinforced concrete shell conceived as $1/8$ of a sphere having a radius of approximately 113 feet (Fig. 1). The main support for the shell is provided

by three reinforced concrete buttresses (or abutments), each located approximately at the north, south and west sides of the structure. Reinforcement of the free edges of the shell is furnished by three reinforced concrete edge beams which span between adjoining buttresses.

The thickness of the shell is 3-1/2 inches, except for a 5-foot wide area along each edge beam, which is thickened to 5-1/2 inches. Also, the thickness of the shell at each buttress is increased to 20 inches. The underside of the shell crown (top of shell) is positioned approximately 44 feet above the Auditorium floor.

Each edge beam spans between adjoining buttresses and has an overall curved length of 173 feet. The depth of each beam above the shell is 14-1/2 inches, while the width is approximately 9-1/2 inches. Secondary supports for each beam and, therefore, for the shell are provided by three structural steel mullions located near the beam center. The crown of each beam is 27 feet above the ground.

Each buttress is a massive reinforced concrete element which extends below the basement of the Auditorium. The attachment of the shell to each buttress is by means of a structural steel pin assembly. This pin assembly permits the shell to rotate in a vertical direction at the buttress. Horizontal movement and sliding of the shell at the buttress are prevented by the assembly.

INSPECTION

Visual Inspections of the Roof

Visual inspections of the structure in 1973 and 1978 had revealed (Fig. 2) deterioration of the roof edge beams in the vicinity of the three abutments. The deterioration (Fig. 2) consisted primarily of "leaching" of the cement paste resulting in the disintegration of the concrete in these areas. In addition, hairline cracking of the edge beams was observed. The damage to the edge beams at the west abutment was more severe than that observed at the other two abutments. A continuous flow of water was also observed at these locations. It was thought that the constant flow of water was caused by condensation beneath and/or leakage through the lead roof covering. The water apparently ran down the shell and escaped from beneath the lead flashing at the three corners of the roof.

During the previous inspections, the condition of the shell structure could not be determined since it was covered



34 Cohen, Dobbs, and Combs

entirely by the roofing system. However, windows cut into the lead covering consistently revealed the roofing materials to be moist. These findings together with the wetness and dampness evidenced at the abutments demonstrated the need for a detailed in-depth inspection of the roof structure. Both interior and exterior surfaces of the shell were inspected for deterioration and leakage. The results of the inspection are illustrated in Figure 3 for the roof slab and in Figure 4 for typical damage sustained by the edge beams.

Inspection of the roof interior revealed the presence of three cracks in the surface. These cracks are shown as wavy dotted lines in Figure 3. The three cracks varied in length from a minimum of 5 feet to a maximum of 12 feet. There were stains on the roof around the cracks which indicated that they had separated over a period of time thereby permitting moisture to penetrate through the shell Figure 5(a). Similar strains or leakage marks were observed at several of the cold and/or construction joints in the roof. Locations where such water damage was observed are shown in Figure 3 and a photograph showing the leakage stain around one of the cold joints is provided in Figure 5(b).

The entire surface of the roof could not be inspected because it was covered with the membrane vapor barrier. Instead, the vapor barrier was removed in a pattern of concentric circular strips starting at the center point at the top of the building. Each strip was approximately 1 foot in width. The concentric strips were spaced at 5-foot radial increments.

The removal of the membrane was accomplished by first scraping the material off the concrete and then sandblasting the scraped area. If a crack was discovered, the membrane was removed over its entire length. Since the exterior of the roof was, in effect, only partially inspected, it is quite possible that other cracks exist in the concrete which were not detected.

A typical example of how cracks were exposed is illustrated in Figure 6.

The inspection revealed several hairline cracks in the roof that were scattered over the surface at arbitrary locations as shown in Figure 3. A much more serious series of parallel cracks was found in the vicinity of each buttress. Each group of cracks, as shown in Figure 3, originated at the edge of the thickened roof, in the vicinity of the buttress, and extended upward in the shell parallel to an axis running

from the lower corner through the center of the structure. These cracks appeared to be wider and longer at the west buttress.

The conditions of the edge beams in the vicinity of the buttresses are illustrated in Figures 4 and 7. The two-edge beams at the west abutment were by far in the worst condition. Approximately 15 feet of deteriorated concrete were removed from both of these members thereby exposing the steel reinforcement. Shoring was ordered to be placed immediately thereafter. Lime deposits produced by the "leaching" of the concrete were found around the buttresses; the support fittings were also found to be rusting.

Test Program

A test program was undertaken in support of the inspection for the purpose of assessing the general condition of the concrete in the edge beams and shell (in the vicinity of the abutments). The test program consisted of non-destructive testing, using the pulse-echo method, together with chemical and petrographic examinations of samples of concrete extracted from the building. Removal of concrete samples was performed by Herman Protze, Inc., under contract with MIT. Performance of the non-destructive tests and evaluation, and the petrographic examinations were performed by Muenow and Associates and Concrete Technology Laboratory, of the Portland Cement Association, respectively.

Locations and sizes of cores taken are shown in Figure 8.

The core extraction proceeded with the removal of the core from the shell located 5 feet up from the west buttress. Attempts to locate the reinforcement in the area, using a James pachometer, proved unsuccessful and the extraction proceeded without fixing the position of the steel. Initial attempts at drilling into the concrete (Cores C6 Top and C6D in Fig. 8) were unsuccessful because the steel reinforcing bars were found to be positioned at 4 inches on center and approximately 2-1/2 to 3 inches below the top surface of the concrete. Therefore, two 3-inch diameter cores were taken from the shell (Cores C6 Bot and C6B in Fig. 8) in lieu of the 4-inch core.

After extracting the necessary cores from the shell, an attempt was made to take a 4-inch diameter core from the inside edge of the edge beam at a distance of 4 feet up from the west buttress (Core C1 Top in Fig. 8). This attempt was abandoned when the cutting tool struck reinforcing steel 4 inches below the top of the concrete. A second attempt was

36 Cohen, Dobbs, and Combs

made using a 3-inch diameter core barrel in the vicinity of the first hole (Core C1 Bot in Fig. 8). However, upon removal of this core, it was discovered that a reinforcing bar had been cut approximately 1 inch below the top of the hole.

The remaining cores (Cores C2 through C5 in Fig. 8) were extracted on December 31. Cores C2, C3 and C5 were taken without incident. However, Cores C-4-1 and C-4-2, which were cut 4 to 5 inches deep, broke at 2 to 2-1/2 inches below the surface and showed indications of past fractures with a slight efflorescence at the points of failure.

The repair of all core holes consisted of first removing water from the holes. After this, a neat cement slurry was brushed inside each hole and the hole was patched. The holes were patched using a dry pack mix prepared using Masterflow 773 non-shrink grout. All cores taken were placed in individual plastic bags which, in turn, were packed in a plywood container, cushioned and protected with pieces of 1-inch thick styrofoam. The cores were shipped to the Portland Cement Association for their chemical and petrographic analyses.

Chemical and petrographic tests--The extraction of concrete cores from the roof began on December 27. In order to correctly perform the associated analysis, it was imperative that each core (1) be extracted in compliance with the appropriate specifications, (2) location be carefully chosen to avoid cutting the reinforcement and (3) void be filled in an acceptable manner.

A total of six cores was removed from the roof. Three cores were taken from the south-west edge beam, two of which were located at distances 4 to 20 feet from the west buttress and the third at the peak of the beam. One core was taken from each of the north buttress edge beams. Each of these cores was removed at a point located 5 feet from the buttress. All cores were taken from the top surface of the roof.

The top and bottom sections of Core C1, which was taken from the west buttress edge beam, were tested for the presence of lead by a comparative chemical method. The precise section of the concrete tested involved that volume containing imprints of reinforcing bars in the highly deteriorated area. The test results indicated the presence of lead, but in the extremely small amounts of 1 part per million. It is the opinion of the PCA petrographer that these amounts were

insignificant and did not reflect any migration of lead ions from the lead roof. Lead, in this amount, would not contribute to the disruption of the cement matrix.

The top and bottom sections of Core C6, taken from the west buttress shell, were also tested for the presence of lead. These tests results, which were similar to those of Core 1, indicated microscopic amounts of lead present. The completed petrographic tests indicated no apparent mechanical disruption of either the aggregates or the cement matrix. No secondary growth patterns were observed which would reflect chemical reactions between the cement and aggregate. Air entrainment in the concrete was minimal with the irregularly shaped entrapped air amounting to between 3 to 5 percent.

Petrographic tests of Core C2 (west buttress edge beam) indicated similar findings to those reported for Core C6. However, indications of initial freeze/thaw damage have been noted in the form of internal microcracking and surface softening of the cement matrix. Traces of alkali aggregate reaction by-products were observed in this core. This is not uncommon in structures of this age. The reaction was deemed non-progressive because the compressive strength of the concrete in this area was equal to, or in excess of, 4,000 psi. Also, the amount of alkali aggregate reaction by-products found was in extremely small quantities and was not found in any of the other cores.

Petrographic tests of Core C3, from the west and south buttresses indicates sound concrete with neither any observed cement nor aggregate description. On the other hand, tests of Cores C4 and C5 (north buttress edge beams) indicated that initial freeze/thaw damage had occurred in the form of internal microcracking and surface softening of the concrete matrix. No secondary growth by-products were observed.

In general, it was concluded from the petrographic tests, that only minor mechanical disruption of either the cement or aggregate occurred in the roof slab and damage by freeze/thaw effects was minimal.

Non-destructive tests--The pulse-echo method of concrete non-destructive testing was used for the evaluation of Kresge Auditorium. This testing method is based upon Snell's laws of reflection. Simply stated, when mechanical energy is introduced into a material, it will propagate until the wave front encounters a material of lower density, at which time, a specific amount of the energy will be reflected back towards the source. The amount of energy reflected depends upon the ratio of energy wave propagation velocities between the two

38 Cohen, Dobbs, and Combs

materials. That portion of energy which traverses the first reflecting interface continues to propagate until another change in density is encountered. This density change may be in the opposite face of a member, in which case, a "black face" reflection is identified (Fig. 9). Density changes of smaller degree will identify such abnormalities as cracking, honeycombing, crushing, etc.

The equipment used with the pulse-echo method includes:

1. Spring-loaded impactor for the creation of a mechanical stress wave.
2. A piezoelectric transducer for converting the mechanical stress energy into electrical energy.
3. A processor for signal enhancement.
4. A storage oscilloscope equipped with a polaroid camera for data presentation.

In order to discuss the results of pulse-echo method testing, a series of terms are defined which will identify the concrete damage recorded by the equipment. These include:

Microcracking - Cracks in the cement matrix with widths up to 0.005 inch and generally non-connecting.

Cracks - Separations in the body of the concrete which may or may not traverse aggregates with widths in excess of 0.005 inch. These separations may be singular in nature or connected with other separations.

Surface Deterioration - Softening of the surface cement matrix, aggregate disruptions, surface-to-rebar depth cracking and surface straining.

Partial Cement Matrix Disruption - Condition may contain some or all of the above-mentioned but will generally be limited to that volume of concrete surrounding complete matrix disruption. Cracking will have progressed to a point that volume changes are visual. Aggregates are loose in their matrix sockets.

Figure 10 graphically illustrates the above definitions.

The non-destructive testing of the Auditorium was performed in two phases. The first phase of tests were performed the weekend of November 23, 1979 (Thanksgiving) and included the testing of the edge beams on the three sides of

the building. The second phase of the tests took place during the Christmas Holiday (specifically, December 26 to 28, 1979) and consisted of the testing of the Auditorium shell. The delay between the two phases was dictated by the roofing removal which was completed in early December 1979.

A total of at least 58 horizontal and 22 vertical measurements (29 horizontal and 11 vertical per half beam Fig. 11) were performed on each of the three edge beams. In the lower portion of each edge beam, near the buttress, where the maximum thickness occurs, two rows of four measurements per row were made . As the beam became shallower, the number of measurements per row was reduced to three (5 rows of 3 measurements per row). From a distance of approximately 17 feet through a distance of 25 to 30 feet measured horizontally along the beam from the buttress, a total of two measurements per row were made. Beyond this point, non-destructive testing was not necessary.

The volume of concrete which could be evaluated by the sonic beam divergence testing method along each row of measurements is illustrated in Figure 12.

The edge beam deterioration, as determined from the pulse-echo method measurements, is illustrated in Figures 13 through 15. Each of these figures shows elevations of the two edge beams at a given buttress. The elevations are also illustrated in the above-mentioned figures.

For a distance of approximately 20 feet up the edge beam, Area 1 at the west buttress (Fig. 13), microcracking, cracking and complete cement matrix disruption occurred between the top of the beam down to the shell/beam intersection. Minor microcracking was noted in the portion of the shell under the beam in this lower 20 feet. From 20 to 25 feet up the beam, the cement matrix disruption fades away, internal cracking between the two layers of reinforcement becomes non-existent, and the surface cracking appears to terminate at the first layer of reinforcement. The upper portion of the beam, for a distance of approximately 17 to 20 feet immediately below where the concrete has been removed and for a distance approximately 10 feet up the beam, the concrete strength varies between 1,000 to 2,000 psi. Below this weak layer, the concrete strength increases to 3,000 - 4,000 psi. Further down into the shell, the concrete strength increases to 5,000 psi and beyond 25 feet up the beam, the concrete strength is 5,000 psi, or greater, throughout the total beam depth.

40 Cohen, Dobbs, and Combs

Deterioration in the west buttress, Area 2 (Fig. 13), was similar to that in Area 1. For a distance of approximately 18 feet up the edge beam from the buttress, microcracking, cracking and complete cement matrix disruption were noted down to the beam/shell intersection. Minor microcracking was noted in a portion of the shell under the beam in this lower 18 feet. From 18 to 25 feet up the beam, the cement matrix disruption fades away; no further internal cracking was noted and the surface cracking (visually observed) terminated at/or slightly beyond the first layer of reinforcement. The concrete strength variation of Area 2 was similar to that of Area 1; i.e., for a depth of 2 to 4 inches, the concrete strength was very low (1,000 to 2,000 psi). Below this depth, the concrete strength increased to 3,000 - 4,000 psi and then to 5,000 psi. Beyond 25 feet up the beam from the buttress, the concrete strength was equal to 5,000 psi or greater for the full depth of the beam.

Severe deterioration occurred in the beam of the south buttress, Area 3 (Fig. 14) at a distance of approximately 7 feet up the edge beam. Microcracking, cracking and partial (approximately 75 percent) cement matrix disruption occurred in the top 9 inches. The volume of the concrete under the 9 inches and extending for approximately 20 feet contained microcracking and minor surface deterioration from 1 to 3 inches in depth. For a depth of approximately 3 to 4 inches below the surface and up the beam for 2 to 3 feet, the concrete strength was reduced to 2,000 psi. Below this low strength area and extending approximately 7 feet up the beam, the concrete strength varies between 3,000 to 4,000 psi. Below this section and further up the beam, the concrete strength was greater than 5,000 psi.

The deterioration of the other beam at the south buttress, Area 4 (Fig. 14), was similar to that described above; i.e., microcracking, cracking and partial cement matrix disruption were found in the top 10 inches for a distance of approximately 7 feet up the edge beam. The volume of concrete under the top 10-inch layer and extending upward approximately 20 feet contained microcracking and minor surface deterioration which were 3 inches in depth. The concrete strengths of the various concrete volume were similar to those of Area 3.

For a distance of approximately 4 feet up the edge beam at the north buttress, Area 5 (Fig. 15), microcracking, cracking and partial cement matrix disruption were noted in the top 6 inches of the beam. The volume of concrete under the upper layer and extending upward approximately 15 feet contained microcracking and minor surface deterioration up to

3 inches in depth. The concrete strength of the upper 6 inches ranged between 3,000 and 4,000 psi; while the concrete volume below the upper layer had a strength between 4,000 to 5,000 psi.

For a distance of approximately 5 feet up the edge beam at the north buttress, Area 6 (Fig. 15), microcracking, cracking and partial cement matrix disruption occurred in the top 4 inches of the edge beam. The concrete under this upper layer and extending for a distance of 15 feet up the beam from the buttress contained microcracking and only minor surface deterioration of approximately 2 inches in depth. The concrete strength in the upper layer varied between 3,000 to 4,000 psi; while below this layer, the concrete strength was equal to 4,000 - 5,000 psi.

In addition to the vertical cracks, some horizontal delaminations occurred in the shell at each buttress. The delaminations were of a localized nature, but could have been expected to spread unless repairs were made. The major delaminations occurred in the shell area adjoining the west and south buttresses.

The relative size and location of the delaminations at the west buttress are illustrated in Figure 16. These delaminations are representative of the delaminations in the shell at all three buttresses.

Vertical cracks, some of which had calcium deposits, were observed along the remainder of each beam. These cracks stopped at the first layer of reinforcement. Only limited areas of internal cracking were noted. In general, the concrete in the central portion of the edge beams was of relatively high strength which varied between 4,500 to 6,000 psi.

Vertical cracking through the total thickness of the shell and limited deterioration were noted in the structure shell at each buttress. This deterioration covered an area which spanned between the edge beams for a distance of approximately 20 feet up from the buttresses. The results of the non-destructive testing and visual mapping (Fig. 3) correlated the occurrence of the cracks. The compressive strength in the shell ranged from 4,000 to 6,000 psi, with the compressive strength of more than 80 percent of the shell varying between 4,500 to 5,000 psi.

42 Cohen, Dobbs, and Combs

Monitoring of Roof Deflections

One of the most important activities undertaken during restoration was the daily monitoring of the deflections of the roof structure. These quantities were measured and recorded at a multitude of locations on the roof beginning on September 25 and continuing through to the completion of the project. Deflections of the roof were measured to detect trends in the structure's behavior which would indicate impending failure.

Deflection measurements were taken once in the morning and once in the afternoon at the same time each day. The quantities measured were:

1. The vertical and horizontal deflections of the three peripheral beams. Horizontal measurements were taken in directions parallel and perpendicular to the beams.
2. The changes in slope of the three edge beams.
3. The vertical deflections at the center of the roof.

Initially, measurements of the edge beam deflections were taken at 13 points on the north-south beam, 10 points on the south-west beam and 11 points on the north-west beam. However, with the installation of the edge beam shoring, a more accurate method was chosen for measuring the vertical deflections, and the number of points monitored for this purpose was decreased to 5 points on the north-south beam and 4 points each on the south-west and north-west beams. Slope changes of the edge beams were monitored at only three points per side. The points at which the deflections were measured were uniformly spaced along each edge beam.

The vertical deflections of the edge beams were initially measured using plumb bobs suspended from the roof structure on plastic (monofilament) lines (Fig. 17). These same plumb bobs were also utilized to measure the horizontal deflections of the edge beams as described in the next section. The lines supporting the plumb bobs were attached to the roof in close proximity (within 1 foot) to the edge beams. When the plumb bobs were first hung, their respective elevations above the floor slab were measured and recorded. Then, twice a day (once in the morning and again in the afternoon), the height of each plumb bob above the floor was measured. From these measurements, the deflections of the edge beam were determined by subtracting the daily elevation from the initial elevation of each plumb bob. This method proved to be unreliable for

measuring all but large deflections which would indicate an imminent roof failure. Accurate measurements of small roof deflections were not possible because the elongations of the monofilament lines were of the same order of magnitude as the quantities to be measured. This was not considered a serious problem until the temporary shoring was installed. When the shoring was installed, however, a more accurate method was required to detect the small edge beam deflections likely to occur. Such a system of measurement was instituted on November 4.

The new system of measurements consisted of the use of an engineer's level and a series of scales suspended from the roof to monitor the vertical movements of the edge beams. The scales were located at equally spaced positions along each beam (5 on the north-south beam and 4 each on the other two beams). Each scale was secured to the floor of the building by a steel wire attached to its base. With the scales firmly secured in the desired positions, reference elevations of the level at each scale location were then established. At the same time, base readings for each scale were also taken. These measurements were taken by first sighting the engineer's level on a leveling rod placed in a fixed position (usually on the curb around the periphery of the building) and then sighting it on each scale. With these initial readings thus determined, subsequent roof deflections at each scale location could then be accurately measured in a similar manner. Twice a day, the level would be set up in the vicinity of each scale and first sighted on the leveling rod, to determine the current elevation of the instrument, and then sighted on the scale. The correction factor for the scale reading was then determined by computing the difference between the current and reference elevation of the level. The roof deflection could then be computed by adding the measured scale correction factor to the base reading (for the scale) and subtracting the current scale reading from the result. The displacements measured in this manner are considered reliable to within the accuracy of the scales used.

The measurements recorded during the length of the project indicated that only minimal vertical displacements of the edge beams occurred. This is illustrated by the displacement plots for the edge beams illustrated in Figure 18. Each plotted line in this figure represents the measured deflected shape of an edge beam on one day. This figure contains several days of plotted measurements for the edge beams. The letters at the ends of each plot designate the location; i.e., N designates "North Buttress".

44 Cohen, Dobbs, and Combs

The horizontal deflections of the edge beams were measured using plumb bobs suspended from the ceiling. Displacements parallel and perpendicular to these members were measured at 13, 10 and 11 locations on the north-south, south-west and north-west beams, respectively. When the plumb bobs were initially hung, their initial horizontal positions were marked on the floor directly beneath them. With the initial position of each plumb bob thus located, the deflections of the edge beam were measured twice daily using a mirror with two orthogonal lines scribed on its reflective surface. The lines intersected at the center of the mirror where a small circle was scribed and reflective substance removed. Horizontal deflection measurements were taken by placing the mirror on the floor of the auditorium and aligning the circle at its center with the mark on the floor indicating the original position of the plumb bob. The inscribed axes (lines) were oriented properly by setting one of them perpendicular to the steel channel cast into the peripheral curb of the floor. Then the horizontal deflection in one of the axes could be measured by sighting along the other axis and measuring the distance from the reflection of the pointed end of the plumb bob to the intersection of the axes. Such a system of measuring the horizontal deflections was utilized to eliminate any parallax errors that would occur if the plumb bob was sighted along the wire supporting it.

Horizontal displacement measurements began in late September and continued until completion of the project. The measurements taken prior to the installation of the shoring reveal that the roof structure was spreading outward (in the direction perpendicular to the edge beam) in the vicinity of the west buttress. This can be seen from the plots of the deflected shapes of the edge beams for the weeks of September 29 and October 29 (Figs. 19 and 20). In these plots, the "x" and "y" axes designate directions parallel and perpendicular to the edge beams, respectively. It is believed that the apparent spreading of the roof was caused by the removal of 18 feet of concrete from each of the edge beams at the west abutment. The plots also indicate that the north-south beam underwent minimal horizontal displacements during the monitoring period. After the shoring was installed, the spreading of the shell at the west abutment was abated.

Daily measurements were taken of the angle between the outward face of the edge beam and a line perpendicular to the ground. These quantities were measured at three equally spaced locations on each beam. It was thought that an imminent failure of the roof would be discernible by large changes in these measured angles.

The readings taken indicate that there was little change, if any, in the slopes of the three edge beams. However, the method used to measure these angle changes was considered unreliable and these measurements were eventually discontinued around the beginning of November. It was thought at the time that sufficient horizontal and vertical deflections were being taken to detect any unusual behavior of the structure.

The vertical deflections at the center of the roof structure were monitored using a tape measure hung from a sleeve at the center of the structure. The self-weight of the tape case maintained sufficient tautness in the tape. The deflections were measured using an engineer's level. The procedure used to measure the vertical edge beam deflections was applied to measure the deflections at the center of the roof. These measurements were taken twice a day and continued through the duration of the project.

In general, the measurements taken reveal that no significant vertical deflections occurred at the center of the roof during the entire monitoring period. All deflections measured, whether upward or downward, were within 1/8 inch of the initial measurement taken at the beginning of the project.

STRUCTURAL ANALYSIS

Structural analyses were performed on the Kresge Auditorium concrete shell roof structure to ascertain the effect of replacing the roofing and performance of the structural repairs. The objectives of these analyses were to define the integrity of the structure for both the duration of the removal of the roofing system and for future occupancy of the building.

The structural analyses of the Auditorium utilized a finite element method solution. Two analyses were performed. The first analysis, which was the more approximate of the two, was performed to determine an estimate of the stresses in the existing structure which, in turn, permitted a determination of whether the existing roof could be removed without endangering the stability of the structure. The second model, which was more detailed, was performed to determine more accurately the stresses which were "locked" into the structure after all repairs were made and the roof was replaced.

The first analysis assumed that the building was "structurally" symmetrical; whereas the second analysis considered that, due to deterioration, the structure was only symmetrical in part.

46 Cohen, Dobbs, and Combs

The mathematical model used to represent the structure in the first analysis consisted of a one-sixth triangular part of the roof (Fig. 21). This model was used because of the six-part symmetrical nature of the roof. The three boundaries of this model consisted of the two edges along the axes of symmetry and one free edge representing one-half the edge beam.

The second model of the Auditorium, as shown in Figure 22 involved a refined version of the one-sixth model and it considered all loadings applied to the structure since the completion of its construction in 1954. The refined one-sixth model was then duplicated, forming three identical models, each having experienced identical loadings histories. The three separate models were coupled along their common boundaries to form a one-half model of the structure (Fig. 23). This expanded model incorporated all of the effects associated with the deterioration of the edge beams, as well as the subsequent roofing system removal, structural repairs and replacement of the roof. The final verification of the structural integrity of the Auditorium was based on this second analysis.

The computer analysis of the Auditorium utilized the finite element program ANSYS. The ANSYS computer program is a large scale, general purpose computer program for the solution of several classes of engineering analysis problems. Analytic capabilities for static analysis include, but are not limited to, creep, elasticity, plasticity, small deflection and large deflection analyses. The matrix displacement method of analysis based upon finite element idealization, is employed throughout the program. In this analysis, the structure being analyzed is mathematically approximated by a finite set of discrete structural elements having known force-displacement relationships at their nodal locations; i.e., points of connectivity. The final one-half roof model of the Auditorium involved a total of 948 finite elements connected at approximately 600 unique nodal locations. The force-displacement relationship for the entire structure is then formulated and solved. The ANSYS program uses the main front (or "frontal") direct solution method for the system of simultaneous linear equations developed by the matrix displacement method. Two elements were used to model the Kresge Auditorium shell; i.e., STIF 44 and STIF 48 (Ref. 1 and 2).

A summary of the sequence used for the computer analysis is given in Table 1. A gravity force equal to one g was applied for all iterations of all steps during the analysis. The large deflection option was also activated in the program

and maintained for all iterations. Each step in the analysis involved a set of solutions (iterations) based on a single set of input force, pressure, displacement, temperature, time and acceleration conditions.

The results of the first analysis are summarized in Tables 2 and 3 for selected critical sections during the loading history of the structure. Table 2 includes a list of shell stresses which occurred just before the structural steel mullions were installed (April 28, 1954); whereas Table 3 gives the stress levels in the shell prior to the beginning of repairs (September 1979). It may be noted that the capacities, as listed in these tables, are based on a yield stress of 36,000 psi for structural steel, a compressive strength of 3,750 psi for concrete, and a yield stress of 40,000 psi for reinforcement steel.

A review of Table 2 indicates that all structural steel, concrete and reinforcement were stressed below their yield capacities. The maximum value of the ratio of the applied loads to the capacity of the elements occurred just beyond the embedded structural steel beam and at edge beams where the haunch moment and splitting moment were equal to approximately 75 percent of their yield strengths. All other stresses and imposed loads were well below their yield strengths.

The results listed in Table 3 were used to make the decision as to the extent to which the structure had to be shored before the repair phase of the project could be started. Here, a comparison of the effectiveness at the lower sections (near the buttresses) of the edge beams was considered.

Stresses in both the concrete and reinforcing steel, as determined from the second analysis, are listed in Tables 4 and 5 for selected critical sections of the structure. Table 4 lists those stresses which would occur after all repairs were made and the new roof had been installed; whereas Table 5 lists the increased stresses due to snow and wind loads. It may be noted that the compression strength listed on these two tables is 5,000 psi, as determined from the non-destructive tests of the structure. Both tables include the effects produced by a 50 percent increase of the load reaction applied to the interior of the shell (acoustical clouds, catwalks, etc.).

As seen from Table 4, the stress levels in all elements were well below the shell capacity, except for the first listing of the section described by Nodal Points 253 to 263. These nodal points describe the portion of the shell located