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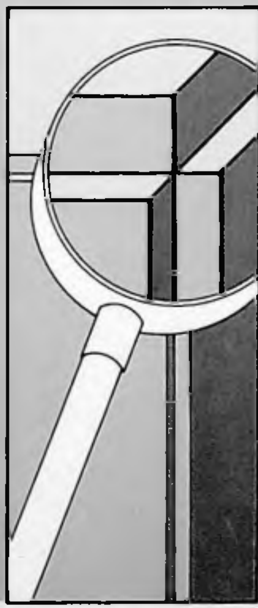
Rehabilitation Guidelines 1982

9
Guideline for Structural Assessment



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- U.S. Conference of Mayors
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- National Housing Rehabilitation Association
- National Home Improvement Council
- Building Code Action
- Council of American Building Officials
- National Conference of States on Building Codes and Standards
- National Fire Protection Association
- American Institute of Architects

FOREWORD

Our purpose in publishing the rehabilitation guideline series is to encourage the rehabilitation and conservation of our older building stock. By making our existing stock safe, sound, and functional, we can very significantly aid in achieving our national housing goals and revitalizing our urban areas. We are emphasizing and encouraging rehabilitation and conservation because they represent the most cost-effective way to add to and maintain our Nation's housing supply.

For some time, we have known that building codes which were established (primarily) for new construction actually served to impede rehabilitation projects. These new guidelines were developed so State and local officials could voluntarily adopt and use them in conjunction with existing codes in the inspection and approval of rehabilitated properties.

This guideline describes the process of gathering and analyzing data about the building and its structural components. Included are detailed descriptions for examining written documentation and conducting on-site surveys, load tests and structural analyses. I believe that this addition to the guideline series will prove useful to many who are involved in the building rehabilitation process.

Samuel R. Pierce, Jr.
Secretary
U.S. Department of Housing
and Urban Development

The Rehabilitation Guideline Series

The *Rehabilitation Guidelines* were prepared by the National Institute of Building Sciences for the Department of Housing and Urban Development in response to the requirements of Section 903 of the Housing and Community Development Amendments of 1978.

As Congress intended, the *Rehabilitation Guidelines* are not a code, nor are they written in code language. Rather, they are designed for voluntary adoption and use by States and communities as a means to upgrade and preserve the nation's building stock, while maintaining reasonable standards for health and safety. The term "rehabilitation", as used in the guidelines, includes any set of activities related to the general view of existing buildings as a resource to be conserved, rehabilitated, or reused.

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

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

2. The *Guideline for Municipal Approval of Building Rehabilitation* examines the inherent differences between regulating new construction and regulating rehabilitation, and presents

specific recommendations for dealing with rehabilitation within municipal building departments.

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

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

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

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

6. The *Electrical Guideline for Residential Rehabilitation* outlines procedures for conducting inspections of electrical systems in existing buildings, and presents solutions to common problems associated with electrical rehabilitation such as eliminating hazardous conditions, grounding, undersized service, number of receptacle outlets, and incompatible materials.

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

extending existing DWV systems, and installing new DWV systems in existing buildings.

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

Because of the value to the building rehabilitation community provided by the initial eight *Rehabilitation Guidelines* published in 1980, two additional guidelines were developed as the *Rehabilitation Guidelines, 1982.* As with *Guidelines 4 through 8, Guidelines 9 and 10* are technical in nature and are intended for use by those involved in the building rehabilitation process. These guidelines cover the following topics:

9. The *Guideline for Structural Assessment* addresses the methods and approaches used to evaluate structural systems in existing buildings. This guideline describes the assessment of common building structural systems such as masonry bearing walls, and simple wood, steel and concrete frames.

10. The *Guideline on the Rehabilitation of Walls, Windows, and Roofs* describes typical methods and procedures and appropriate cautions attendant to rehabilitating many common examples of these building components for many existing structures.

The *Rehabilitation Guidelines*–or copies of specific guidelines are available from HUD USER, P.O. Box 280, Germantown, Maryland 20767. Phone (301) 251-5154.

Contact HUD USER for cost and ordering information.

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Acknowledgements

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Introduction

In many instances, the assessment of the condition of an existing building's structural system is the key to determining the building's continued use or rehabilitation. This guideline addresses the assessment of common building structural systems such as masonry bearing walls and simple wood, steel, and concrete frames. However, the assessment process and methods of test described in the guideline apply, in principle, to all structural types and combinations.

This guideline is intended for use by persons experienced in structural assessment. It also will provide general guidance to others with interest in, although not experienced in, structural assessment. The structural calculations outlined in the guideline should be performed only by qualified professionals, and all field and laboratory testing should be carried out by specialized technical experts.

The structural components of an existing building are subject to the building code requirements for new construction in two circumstances:

- Repair and improvement of an existing building when code compliance is triggered by a "25-50% rule" or similar provision. See Guideline 1 for a detailed explanation of the 25-50% rule.

- Change of use or occupancy classification when code compliance is triggered by a "change of occupancy" provision.

While there is no absolute measure of adequate structural safety and serviceability, current building code requirements represent a generally accepted minimum level of structural performance that should be used as baseline. However, experienced and reasoned professional judgment should temper literal compliance with code provisions for new construction, since these requirements are based on materials and methods that may not have been used in buildings constructed in an earlier time.

When an existing building is unaltered or its use is unchanged, its structural components are subject only to the requirements of building maintenance codes (usually issued as property maintenance, housing, or health codes), hazard abatement codes, or retroactive laws and regulations. Building maintenance codes generally require that the structural safety features of a building be maintained at a level of performance comparable to that existing at the time the building was constructed. Hazard abatement codes state the absolute lowest level of structural safety that can be permitted in an existing building (see Guideline 1, page 57), while retroactive laws and regulations require that all buildings of a certain class or occupancy be altered to eliminate a specified unsafe condition. To date, retroactive laws pertaining to structural safety have been enacted in regard to the seismic reinforcement of buildings (see Guideline 1, Appendix 8), for exterior masonry walls (see Appendix D of this guideline), and in a few instances, for structural certification in general (i.e., Dade County, Florida).

Appendix A of this guideline gives a brief historical overview of the evolution of building code provisions for structural safety.

1 Overview on Structural Assessment

1.1 General

The fundamental purpose of structural assessment is to confirm that a building is structurally safe for its existing or proposed use. Structural assessment is a different activity from structural design--it is aimed at evaluating the actual condition of a building in-place and with a history of use.

A structure that has withstood the combined service loads and environmental stresses placed upon it over many decades has in effect proved itself. But the possible deleterious effects of these forces should be considered during the structural assessment process. In many cases the loads and stresses experienced by a building are different from those foreseen at the time of design and construction or envisioned in the building regulations, if any, under which it was built.

Loads and environmental stresses that must be considered in the structural assessment process include:

- Dead Loads. Changes in dead loads from those anticipated in the structural design calculations may arise from variations in the dimensions, density, or moisture content of building materials, the substitution of building materials during construction, or from later alterations and additions. Extensive partitioning of a previously unpartitioned loft building, for example, can increase the building dead load significantly.

- Snow and Ice Loadings. Consideration should be given to the potential for build-up of snow and ice, particularly in roof valleys. Climatological records may indicate unusual snow loadings at some point in the building's history that will warrant additional investigation.

- Wind Loads. Climatological records should be checked for unusual wind loads to which the building may have been subjected. A building's ability to sustain high winds depends in part on its geometry and orientation, and such pressure relief mechanisms as windows. Buildings with a history of unusually high wind exposure should be examined with special attention given to structural connections.

- Seismic Loads. Seismic design is a relatively recent science, and few buildings constructed before the 1920s or 1930s were designed to accommodate these loads. Retrofitting structures to meet modern seismic safety requirements can be a major undertaking. Local laws should specify if an existing building must meet retroactive seismic requirements.

- Soil Pressures and Ground Movement. Because soil bearing capacities and building foundation loads vary extensively, differential settlement is common in older buildings. A small amount of settlement will be structurally important only if long term moisture leakage through fine cracks has adversely affected building components. Large movements in contiguous structural elements such as beams, columns, walls, floors, and roof systems must be evaluated for possible distress, including fracture, loss of bearing, or overstressing of joints and fasteners. Monolithic materials, such as concrete and masonry, are generally incapable of accepting such movements and will crack. Such cracks are most likely to occur at corners and large openings. Since, in most cases, differential shears are involved, cracks will typically be diagonal. Adverse effects of ground movement may be caused by:

- failure in loadbearing soil strata
- soil consolidation, shrinkage, and swelling
- heave due to frost
- earth tremors, earthquakes
- soil compaction due to vibration
- movement due to construction on the site or in the locality
- soil erosion from faulty drains or water supply
- change in water table
- plantings
- inadequate foundation design or gradual foundation deterioration

Service Loads. During its service life, a structure may have been exposed to a variety of loadings associated with building use. These commonly include machinery, storage, hoistways, or the like that may induce local structural stress and long term deformation or weakening. The history of the imposed loads to which a building has been subjected should be investigated as fully as possible. If the building has been used for manufacturing, detailed investigation and special expert advice may be called for.

Environmental Effects. The effects of daily and seasonal changes in ambient temperature in a structure cause expansion and contraction. Extremes of temperature may affect the performance of materials. Variations in moisture content of some materials will also cause expansion and contraction of a possibly deleterious nature. Vacant or abandoned buildings normally experience the greatest temperature and humidity

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extremes, and if unoccupied for long periods of time, should be investigated with special attention to the problems associated with these conditions.

- Creep. Creep, the slow change in dimensions of a material under stress, must be considered when evaluating structural components. The level and duration of stress, and the properties of the material itself, are the determining factors in creep evaluation. Of particular concern is horizontal frame shortening in reinforced or post-tensioned concrete.

- Effects of Vibration. Vibration from machinery within the building or vibrations from passing traffic (especially trains) can have deleterious structural effects. Blasting vibrations from nearby construction operations can cause potentially serious building problems. Vibrational distress is usually manifested in cracked plaster, missing mortar, and sometimes fatigue (particularly in metals) if the number and amplitude of vibration cycles are excessive.

- Fire. The effects of fire on the structural components of a building vary according to material and exposure. Appendix B contains a detailed explanation of this subject. It should be noted that structural modifications may considerably reduce the fire resistance of the structure, and that all such modifications should be considered in relation to building code requirements.

- Abrasion, Erosion, and Corrosion. Exposed building elements are subject to abrasion and erosion by weather, vandalism, equipment, cleaning materials and agents, and general use. Sources of corrosion include moisture, chemical cleaners, airborn pollutants, cathodic reactions, fire retardants, and chemicals associated with manufacturing uses. All can be structurally damaging, although corrosives are potentially the most damaging. Fire retardants used in wood and cellulostic insulation can be especially damaging to metal structural connectors when moisture is present. Different metals separated on the galvanic scale but in close contact will corrode in the presence of water or other electrolytes.

- Impact. Impacts caused by routine equipment usage and those associated with unusual forces (such as traffic accidents and explosions) can seriously damage structural supports and other building components. Over a long lifetime a building may be subjected to a number of such damaging forces. During the structural investigation, special attention should be given to exposed columns and their connections in areas where there is evidence of equipment usuage or vehicular traffic.

- Flooding. Both short and long term structural damage may be caused by flooding, including erosion of the soils supporting the structure, unusual lateral forces or impacts, and materials deterioration caused by excessive moisture. The structural investigation should include the uncovering and examination of all structural members and connections exposed to flooding.

- Fungal and Insect Infestation. All organic building materials are susceptible to fungal and insect infestation. Wood structural members will not normally be subject to attack if kept in a relatively dry condition. Completely dry wood will never decay, and the drier the wood the less likely it is to be attacked by most types of wood-inhabiting insects. A complete description of the types of fungal and insect infestations that attack wood is given in Appendix C.

1.2 The Assessment Process

The structural assessment process consists of gathering and analyzing, in a systematic fashion, data about the building and its structural components. The four general parts of this process consist of 1) examination of written documentation, 2) the on-site building survey, 3) laboratory and field testing, 4) structural analysis, and 5) load testing.

1) Building Documentation. Valuable information on the design, construction, and history of the structure itself may be available from documents governing or prepared for the original design and construction and for subsequent modifications. It should be noted, however, that drawings and calculations may not be accurate or reflect as-built conditions, or subsequent modifications and/or additions.

   Possible sources of building documentation include:

   - building owner or staff
   - local building department drawing or microfilm files and inspection reports
   - design professionals (architects, structural engineers) and contractors associated with the original construction or subsequent modification of the structure (or their successor firms)
   - local library, historical society, newspaper files
   - professional publications dating within a year of the building's construction or subsequent major modification(s)

- local codes in effect at the time of construction

2) <u>Building Survey and Field Testing</u>. In general, unless there is obvious overloading or a significant deterioration of important structural elements, there is usually little need to verify the original structural design. The effects of time on the original construction materials should be first evaluated by a thorough visual examination of the building, supplemented by field and laboratory testing when needed.

a. <u>Visual Examination</u>. A visual examination will in most cases be adequate when executed systematically by a person experienced in structural inspection. Surface imperfections such as cracks, distortions, sagging, excessive deflections, significant misalignment, signs of leakage, and peeling of finishes should be viewed critically as indications of possible difficulty. Randomly located deficiencies complicate the diagnosis; therefore, from the initiation of the investigation, it is important to be sensitive to the identification of a patterned series of deficiencies. Since a significant indication of potential structural problems is the crack, the location, magnitude, and extent of cracking should be carefully noted and studied. Chapters 2, 3, 4, and 5 list common visible defects and recommended investigation techniques for wood, masonry, metal, and concrete structural systems.

Because the most widespread and serious deteriorating agent of building materials is water, the building's exterior walls and roof should be carefully examined, and the presence of water and its path into the structure documented. The roof should be free of excessive deflection and its covering should be watertight. Flashings, gutters, and drains should be in good condition and capable of directing water away from unprotected building components. Walls, windows, and doors should be reasonably watertight. Common points of moisture entry in walls are cornices, sills, ledges, caps, and copings.

b) Field Testing. Field tests such as chipping small areas of concrete and surface finishes for closer examination are encouraged where visual examination alone is deemed insufficient. Generally, unfinished areas of buildings such as utility spaces, maintenance areas, stairwells, and elevator shafts should be utilized for such purposes. In some cases, ceilings or other construction finishes may have to be opened for selective examination of critical structural elements. Whenever possible, such locations should be carefully selected to be the least disruptive and most easily repaired. A sufficient number of structural members must be examined to afford reasonable assurance that they are representative of the total structure.

3) Laboratory Testing. Laboratory tests and quantitative analysis will not generally be required for structural members or systems except where visual examination and field testing has revealed such need, where apparent loading conditions may be critical, or where a change in occupancy will increase loading.

Laboratory tests for wood, masonry, steel, and concrete are discussed in Chapters 2 through 5. All laboratory tests should be conducted by a qualified testing laboratory.

Structural Analysis. If visual examination, field and laboratory tests, or other circumstances determine the need for a quantitative analysis of structural capacity, precise and accurate information on the existing structure must be gained from either documentary sources, such as the original building drawings, or from field measurement. All documentary information should be field checked, and the structural analysis should be conducted only by a qualified structural engineer.

a) Initial Analysis. Initial calculations should be made for the structure's load carrying capacity and margins of safety, using available information on actual loads and on the size and strength of materials and components. In particular, the structure's inherent stability and adequacy for its intended function should be examined. It will be necessary to make assumptions about the distribution of loading and the strength of materials. Such assumptions should be conservative, and follow the practices recommended by the relevant professional engineering or standards-making organizations.

The initial calculation will practically always be a conventional structural analysis. This may lead to one of three possible conclusions:

(i) The calculations show that the structure has an adequate margin of safety according to current code requirements for its intended use.

(ii) The calculations indicate that the

structure is grossly overloaded to the extent that the calculated overall factor of safety is unity or less. If the structure nevertheless is carrying its load without any signs of overstress and generally appears in good order, the basis of the calculation must be examined for error.

(iii) The calculations indicate a factor of safety greater than unity but less than that required by the building code, and the structure shows little if any indication of overload. In this case a refined calculation is called for that considers alternative load paths.

b) Refined Analysis. The initial calculations were based on conventional design assumptions. If the calculations indicate a moderate shortfall in load-bearing capacity it may be worthwhile to improve the accuracy of the assumptions by measuring the exact structural geometry and dimensions of members, by measuring the actual thickness of and determining from samples the densities of the materials that make up the dead loads, and by carrying out tests to ascertain the strengths of the structural materials on site or in the laboratory.

With these more accurate data a second series of calculations may be carried out. The most common design calculations use very simplified (mostly 2-dimensional) models, and the mechanical properties of the materials are simulated by fairly coarse approximations since the quality of materials and workmanship is to some degree unknown.

As a consequence, the secondary contributions to the load-carrying capacity of a member, and the reductions in the loads acting on a particular member or part of member (which arise from static indeterminacies), may be ignored.

When structural failures are investigated they are usually found to be caused by a combination of several factors. This should be borne in mind when planning the calculation checks.

5) Load Testing. With the exception of long-term tests involving implanted instruments, the scope of full-scale structural tests is usually very limited. Often it is practical to load only one bay of a building and to apply as few as one or two loading cycles, which for nondestructive tests must be kept well within the elastic limits. The value of such tests may be more for demonstration than for technical purposes. For additional information see ASTM STP-702, Full Scale Load Testing of Structures.

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2 Wood and Timber Structures

2.1 General

Wood and timber structures are especially vulnerable to three general kinds of problems: 1) those associated with shrinkage, deflection, and creep, 2) those caused by infestation of wood inhabiting organisms, and 3) fire. Problems relating to wood inhabiting organisms are discussed in Appendix C, and those associated with fire in Appendix B. Experience indicates that most failures in wood structures occur in individual members or components and do not result in the failure or collapse of the structure. There is usually an elastic or plastic readjustment among other framing members so that stresses are redistributed to other components in the assembly.

2.2 Problems Related to Wood Shrinkage and Deflection

Wood shrinkage makes structural connections the weak links in many wood structures. Visible indications of structural distress are splits in the wood adjacent to bolt holes, elongated bolt holes, abnormally large deflections, loss of bearing, or sagging. Failure to tighten bolts periodically may result in excessive vertical deflection, in bowing and twisting of members, and in split rings partially coming out of their grooves. The National Design Specification for Wood Construction specifies that the load capabilities for bolted and lag-screwed connections with splice plates should be reduced to 40 percent of full design strength for timber that seasons in place.

Long term deflections in older structures, particularly of the industrial type, may have opened important joints, even in the absence of other problems. Typical problems are listed below:

Visible Defect	Possible Cause	Investigation
a. sagging structural members	excessive span/depth ratio of members inadequacy of any main supporting beam or joist change of roof covering material or addition to existing covering inadequate or lack of bracing heavy loadings such as water storage tanks, mechanical equipment, etc. 'ponding' of water on roof defects in the actual structural timber such as wet/dry rot, splits, and insect infestation creep due to use of unseasoned wood	determine size and spacing of members and compare with span check adequacy and condition of supporting beam(s), and that additional layer(s) of asphalt have not been applied since completion of construction check equipment location(s) and adequacy of provisions for its support visual examination for rot or insect infestation
b. ridge sagging	horizontal movement of feet of rafters removal of purlins and/or struts, defective wall in topmost story supporting the feet of the struts carrying the purlins	examine verticality of external walls in the topmost story; extent and efficiency of ties provided between the feet of rafters (i.e., ceiling joists); condition

Visible Defect	Possible Cause	Investigation
	recent change in the roof covering causing an increase in loading	and spacing of purlins and struts; support for struts in topmost story; check roof for recent changes in covering materials
c. valley beam sagging	removal or settlement of central support (usually timber stud partition) valley beam split or rotten where built into external walls or because of leaking gutter or poor flashing insect or fungal damage	visual examination of support provided and its effectiveness remove part of ceiling for visual examination, particularly where beam joins the wall visual examination for rotted wood and wood inhabiting insects, their holes, and wood particles resulting therefrom examine gutter and flashing for points of water entry

2.3 Methods of Test for Wood

The practice of evaluating wood must focus on identification of those characteristics of the wood (such as density, knots, moisture content, etc.) that define its performance because, as contrasted with man-formed materials (such as concrete and metals), timber must be segregated into use classes (graded) according to predicted performance. This means that the building industry's control over the quality of wood is limited to the proper application of these use classes.

In existing structures, questions of performance of wood products must address: 1) the presence of the properly specified product grade and size, 2) the proper joining of these products, and 3) evidence of degradation of these products by the environment (e.g., moisture, temperature, chemicals, etc.).

One of the greatest aids available for evaluating structural timber is the grade mark stamped at the mill. Because grade marks usually can be related to a recommended design value (by reference to the National Forest Products Association's National Design Specification for Wood Construction, or other relevant documents), it is helpful in determining the quality and strength properties of the existing timber structure if they can be discerned. Doing so may require removing the finish on the members, and this is usually not done because of the considerable costs of such explorations compared to the benefits derived.

When grade marks do not exist or are not discernible, it may be necessary to engage an evaluator experienced in identifying and grading wood products to make the evaluation of the quality and strength properties of the structural members in question. In order to accomplish this task, many nondestructive and destructive evaluation procedures are available, the most common of which are listed below.

a. Visual Examination for Strength and Grade. For detecting such qualities as sizes and frequency of knots, grain slope, and may lead to a stress reduction factor (compared to stress value of clear wood). It is well suited for grading inspection. Provides a measure of structural adequacy related to conventional practices under accepted ASTM/ALS grading practices. Limited to accessibility. May be impractical if wood is covered with paint.

b. Penetration Test (Pilodyn Penetrometer). For estimating approximate in-situ strength properties and degree of decay. Equipment is portable, simple, durable, and can be used by field personnel with appropriate training. Does not provide a precise determination of strength. Readings must be calibrated with known samples. Cannot measure decay unless it proceeds inward from the surface. Measures only advanced stages of decay.

c. <u>Moisture Meter</u> (dialectric type). For determining moisture content. <u>Capacitance meter</u>: measures a change in oscillation frequency due to moisture content/dielectric constant of the wood or change in the capacitance of the electrode as an impedance element when in contact with the specimen. <u>Power-loss meter</u>: measures a loss of amplitude of an electrical wave emission resulting from amount of moisture in wood. Both types are easy to use. There is no physical disturbance of the surface. Useful range of the dielectric type moisture meters is from 0 percent to appromimately 30 percent moisture content. Sensitive principally to the surface of the sample. Accuracy is relatively low, particularly when moisture gradient is present. Readings are affected by specimen density, chemical treatments, or decay.

d. <u>Moisture Meter</u> (resistance type). For determining the moisture content of any size piece of lumber by measuring its electrical resistance between two probes inserted in the lumber. Equipment is simple and rugged. Readout is in direct units, calibrations available for various grades and species. Yields approximate results in only the 7 percent to 30 percent moisture content range. The data are influenced substantially by some preservatives, fire retardants, and decay.

e. <u>Stress Wave Propagation</u>. For determining strength and modulous of elasticity. The propagation or velocity of transverse stress waves is influenced by inconsistencies in the wood that may affect strength. The density and wave velocity in a sample are measured to yield a modulus of elasticity. Strength evaluations are based on that value. Portable, light-weight, low cost. Requires trained personnel to operate.

f. Pulse Velocity. For measuring strength and modulus of elasticity. Major variations in the velocity of a longitudinal wave propagation indicate possible discontinuities. The transmission time for a longitudinal wave propagation is measured and then translated into modulus of elasticity that is used to estimate degree of decay and to assist in determining strength from tables of sound wood. Equipment is portable and readily adaptable for field use. Relatively fast measurements. Velocity can be affected by wood characteristics that are not flaws (such as moisture content), reducing accuracy.

g. Weight Test. For measuring moisture content. Samples of wood are taken from a structural member and differentially weighed to determine moisture content (before and after drying). Accurate results can be expected at any level of moisture content. Requires laboratory test equipment. Takes considerable time.

h. Radiographic Test. For determining grain direction, irregularities, decay, splits, knots, moisture content, insect damage, location and size of members in a floor or wall system. Capable of measuring thickness variations over four percent in 1/2 inch thick materials (10 percent in 1 inch thick materials). Can detect internal density variations. Provides a permanent record. Equipment for wood evaluation is lightweight, portable. Test is easy to perform. Radiation is harmful to living tissues and operators must be shielded. High initial cost. Time delays for film development. Must have access to opposite sides of test specimen.

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3 Masonry Structures

3.1 General

Masonry structural systems include loadbearing walls and piers, nonbearing panel and curtain walls, and shells. Types of nonreinforced wall construction include: monolithic or composite solid walls of brick or concrete masonry units; metal tied cavity walls; masonry bonded rib walls; masonry bonded hollow walls; and masonry veneers over studs of steel or wood.

Before the advent of iron and steel framing in the second half of the 19th century, masonry was the only available material for high-rise buildings. The absence of a rational structural design theory for masonry resulted in very thick masonry walls, reaching 6 ft thick at the base of a 200 ft high loadbearing masonry building. After the 1890s, masonry exterior walls in high-rise buildings were mostly confined to relatively thin, nonbearing, curtain walls about one foot thick, supported at each floor. Shortly after World War II, the Swiss developed a rational structural design theory for masonry, which spread across Europe and by the 1960s had reached the United States. For many types of high-rise buildings, frameless loadbearing masonry walls have now become common at heights up to 22 stories.

Until about 1890, the standard mortar for masonry was a mixture of sand and pure lime (i.e., hydraulic lime) or lime-pozzolan-sand. These low strength mortars gave masonry a low modulus of elasticity and, therefore, an ability to absorb considerable strain without inducing high stress. Accordingly, the tendency to crack was reduced, and when cracks did appear, masonry of high lime mortar was to a great extent capable of chemical reconstitution, i.e., "autogenous healing."

3.2 Structural Problems of Masonry

Structural damage to masonry walls may be caused by overloading, ground movement, thermal or moisture movement, shrinkage, fire, roof or floor movement, lateral loads, sulphate attack, corrosion of embedded metal, frost, salts, and unsound materials. Strength and stability may be impaired by even small wall bulges, and are especially critical in cavity walls and those built with hydrated lime mortars.

In general, points of high shear and low moment should be identified and examined since joints, bearing surfaces, and connections are the most vulnerable and critical areas of a masonry structure. Bulging, sagging, or other signs of misalignment may indicate related problems in other structural elements. Vertical and horizontal cracking where masonry abuts columns or other frame elements such as floor slabs should be critically examined. The behavior of masonry in a reinforced concrete frame is likely to differ from that of the concrete with respect to volume changes due to variations in the moisture content and in ambient thermal conditions. Absence of provision for differential movement between dissimilar materials in masonry structural systems frequently causes cracking and increased water permeance. Brick Institute of America (BIA) Technical Note No. 18 analyzes (and illustrates) problems associated with differential movement and masonry cracking.

Corrosion of metal anchors, ties, and reinforcement embedded in masonry may be a problem unless the metal has a protective coating (such as galvanizing), or unless a masonry cover of at least four inches is provided for steel in walls. Rain and snow may contain carbonic, sulfuric, nitric, or hydrochloric acids that lower the pH of rain water to between 3 and 5, thereby accelerating corrosion. All metal embedded in masonry walls should be checked to the fullest extent possible.*

Typical masonry problems are listed below.

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* For a complete discussion of this problem, see "Diagnosis of Non-Structural Problems in Historic Masonry Buildings," Baird M. Smith, as published in Conservation of Historic Stone Buildings and Monuments, Norbert S. Baer, ed., National Academy Press: Washington, D.C., 1982.

LOADBEARING SOLID BRICK OR BLOCK WALLS

Visible Defect	Possible Cause	Investigation
a. wall out of plumb	rotation of foundation, lack of lateral restraint pressure from horizontal forces overloading faulty construction eccentric loading	examination of foundation and subsoil for presence of tree roots or insufficient depth of foundation adequacy of wall/floor connections and roof ties (in case of pitched roofs)
b. wall fractured vertically -- cracks tapering or diagonal following vertical and horizontal brick joints	differential foundation or support settlement heave in clay soils roof movement overloading cutting of chases lack of control or expansion joints	adequacy of foundation depth of foundation presence of water (broken drains) ground water movement proximity of tree roots
c. wall bulged	inadequate lateral restraint inadequate "bond" overloading horizontal chases thermal movement lack of horizontal expansion joints	adequacy of wall/floor connection use of clip headers, especially in Flemish bonded walls loading and design appraisal
d. walls fractured vertically and/or horizontally--cracks not tapering	movement/corrosion of embedded structural steelwork overloading thermal movement moisture movement	ascertain presence or otherwise of structural steelwork in wall and adequacy or omission of concrete casing check for proper expansion joints
e. splitting and/or bulging or brick piers--fractures tapering off to zero at top and bottom of pier	overloading fault in bond thermal movement lack of expansion joints	loading and design check further field examination

LOADBEARING BRICK OR BLOCK EXTERNAL CAVITY WALLS

Visible Defect	Possible Cause	Investigation
a. bowing of inner or outer wythe	inadequate lateral restraint from floors inadequate tying of wythe overloading inadequate design thermal movement faulty design or construction moisture movement	omission or inadequate number of wall ties, corrosion of ties, or poor strength of brickwork and mortar used for this purpose loading and design appraisal
b. vertical tapering cracks and/or diagonal cracks following horizontal and vertical brick joints in zig-zag pattern	differential foundation movement and/or heave in clay soils differential movement of support	adequacy of foundation depth of foundation below surfaces of ground presence of water (broken drains) ground-water movement due to adjacent construction proximity of trees
c. vertical hairline fractures at vertical joints with some evidence of hairline crack along bed joints (not zig-zag)	shrinkage of mortar lack of expansion joints	evidence of no vertical differential movement, symptomatic of foundation failure

NON-LOADBEARING BRICK OR BLOCK EXTERNAL CAVITY WALLS

Visible Defect	Possible Cause	Investigation
a. bowing of inner or outer wythes	creep, shrinkage and elastic shortening of building frame moisture movement (expansion of brickwork) inadequate tying of wythes thermal movement faulty construction	provision (or otherwise) of 'compression' joints at the top of panel walls and their effectiveness adequacy and condition of ties between wythes, their spacing and embedment

Visible Defect	Possible Cause	Investigation
b. horizontal fractures in side walls at concrete floor levels	thermal movement moisture movement	presence of underfloor heating inadequacy or omission of joint between slabs at expansion joints
c. dampness on inside face of inner wythe	water bridging of cavity condensation inadequate flashing clogged or missing weep holes	existence and size of cavity mortar droppings on ties other bridging of cavity--wood, bricks, etc. omission of weep holes or drainage joints in bottoms of outer wythe bridging of cavity by faulty filling of cavity with thermal insulation material
d. cracking (random pattern) of internal plaster finish	thermal movement moisture movement	shrinkage of blocks due to central heating inadequate mechanical tying of inner wythe to column at edges of panel

BRICK PARAPET WALLS

Visible Defect	Possible Cause	Investigation
a. parapet 'bowed' on plan or leaning inwards or outwards	climatic effects thermal movement moisture movement lack of expansion joints	evidence of similar defects in adjacent buildings in same situation examination of condition of timber plates built into walls look for horizontal cracks at perimeter of building
b. parapet brickwork saturated	faulty construction porous brickwork no drainage holes under coping leaky coping inadequate flashing	check flashing and drainage holes under coping stone (if any) and their condition suitability of bricks used for purpose

Visible Defect	Possible Cause	Investigation
c. diagonal cracks along brick courses and down vertical joints at each end of parapet	moisture movement thermal movement	porosity of wall construction and likelihood of expansion due to absorption of moisture effects of temperature and orientation (i.e., south facing)

3.3 Methods of Test for Masonry

Several nondestructive evaluation methods have been used with various degrees of success to determine the physical properties of masonry units and mortar. However, these test methods have limited application and generally provide information only on the physical make-up of the masonry unit (continuity, location of voids, reinforcement, etc.). In only one case (low frequency ultrasonics) can an estimate of compressive strength be obtained by experienced operators and evaluators, but it is prohibitively expensive to use in routine investigations.

In contrast to the limited choice and undetermined reliability of nondestructive methods for evaluating in situ strength properties of masonry, there are a number of more thoroughly developed methods available for this purpose which are termed destructive or which require testing other than in situ. Most of these tests, however, are designed for testing materials used in new construction, and were not developed for testing existing masonry. Therefore, their use as forensic tests is limited by the experience of the person doing the testing and his knowledge of how to constructively interpret the test results.

a. Probe Holes. For determining location and uniformity of the inner cell grout and wall thickness. Operator penetrates the area of investigation with a small masonry bit and probes the hole with a stiff wire. Small holes may be patched easily. Surface damage is only minor. Fiber opties also may be used in some cases.

b. Low Frequency Ultrasonics. For determining continuity (voids or cracks) for estimation of compressive strength. Soniscope and two transducers are used. Transmitter and receiver are placed on opposite sides of the masonry unit. Low frequency ultrasonic sound waves are transmitted. Travel time and relative strength of transmitted signals is measured. Voids or cracks in units will weaken the signal. Compressive strength can be estimated by correlation of pulse velocity through units and mortar with compressive strength of cores or prism that were removed from the wall tested. This equipment usually is available only through specialized consultants and requires operators and evaluators who are very experienced with testing of masonry. The cost may be prohibitive for routine investigations.

c. Gamma Radiography. For determining location of voids and/or reinforcement. The gamma source and x-ray film are placed on opposite sides of the test specimen. After exposure of the film for several minutes, the film is processed and read. Voids show on the film as dark irregular patches. Reinforcement shows as a light area on the film. Access to two sides of the test specimen is required. Extensive safety procedures are required due to the health hazard of gamma ray exposure. The cost could be prohibitive for routine projects.

d. Pachometer. For determining location of steel reinforcement. The pachometer is a magnetic detector based on the principle that a ferro-magnetic component (such as steel reinforcement) will cause a variation in the magnetic field induced into the masonry (a non- magnetic medium). The surface of the masonry units is scanned with a probe. Readings indicate location, size, and depth of reinforcement. This test is used only for light reinforcement. If both joint and cell reinforcement are used, results are difficult to interpret.

e. Hammer Test. For testing structural soundness of units, bond with mortar. Operator lightly taps the masonry unit with a hammer and listens to resonant sound. A very experienced evaluator might be able to determine the condition by the sound. This test requires an experienced person with a good sense of hearing and a delicate touch. It is an unsophisticated test with questionable results. Test cores may be needed to validate findings.

See Guideline 10, Chapter 1, on cleaning, repointing, and coating problems and solutions associated with exterior masonry walls.

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4
Iron and Steel Structures

4.1
General

Cast iron was the first metal to be widely used as a structural material. It is remarkably strong in compression, and was used throughout the 19th century as the standard choice for structural columns. Cast iron is much weaker in tension, and thus not appropriate for structural beams. Instead, wrought iron, with its greater tensile strength, was used for structural beams. By the late 19th century, builders and manufacturers turned to steel, which was stronger than cast iron in compression and wrought iron in tension.

The transition from iron to steel was gradual, and for years many buildings used both iron and steel. The first completely steel-framed structure was completed in the early 1890s. By the turn-of-the-century, steel frame construction became standard for structures of more than a few stories. Exterior walls were "hung like curtains" on a lighter but stronger frame. A whole new vocabulary of building connections was developed for anchorage of the curtain walls to the skeleton and for the complex connections between columns, beams, and girders.

Although iron and steel are not combustible, they lose strength in a fire if they are not protected from the heat. Almost all structural steel has to be fire protected in some manner, utilizing claddings such as terra-cotta, tile, plaster, concrete, sprayed concrete, gypsum wallboard, or sprayed insulation. Therefore, once covered most structural steel is difficult to assess in later years.

4.2 Structural Problems of Iron and Steel

The deterioration of iron and steel can result from a variety of causes, the most common of which is corrosion. Often called oxidation, it is the chemical reaction of a metal with oxygen or other substances. The deterioration of metal is a complex process because the type and degree of corrosion is affected by minor variations in environment, contact with other metals and materials, and the composition of the metal itself. With some metals, the oxide coating forms a nonporous protective membrane which restricts the passage of metal ions through it. In other cases, such as the rusting of iron, the oxide does not form a protective coating but rather promotes the continued corrosion of the metal.

Iron and steel structural components most likely to corrode are fasteners, welds, and interfaces with masonry or concrete. Column bases should be examined when located in damp locations or where flooding has been experienced, particularly if salt water has been involved. Thin cracks in concrete fireproofing will usually indicate only minor corrosion, while extensive spalling may indicate a much more serious condition.

Steel bar joists are particularly sensitive to corrosion. Critical areas are web member welds, especially near supports, where shear stresses are high and possible failure may be sudden.

Iron and steel can also deteriorate from purely physical causes, including:

- Fatigue. Fatigue is failure of a metal by the repeated application of cyclic stresses below the elastic limit--the greatest stress a material can withstand without permanent deformation after removal of the load. It results from a gradual or progressive fracture of the crystals.

- Graphitization. Graphitization may be a problem in cast iron exposed to acid rain or salt water. It is caused by the impregnation of porous graphite corrosion residue with insoluble corrosion products as the iron corrodes. The cast iron retains its appearance and shape but loses much of its mechanical strength.

- Fire. Fire can cause unprotected iron and steel framing members to become plastic and fail rapidly. Structural iron or steel that has survived a fire without deformation is usually safe to reuse, but any questionable member should be load tested. See Appendix B, "Fire."

- Overloading. Overloading is the stressing of a metal member beyond its yield point so that permanent deformation, fracturing, or failure occurs. It can fail through the application of static loads, dynamic loads, thermal stresses, and settlement stresses either singly or in combination. "Buckling" is a form of permanent deformation from overloading which is usually caused by excessive weight but can also be caused by thermal stresses. Members can also be overloaded if their support is removed and loads are redistributed to other members which can become overstressed and deformed.

- Connection Failure. The failure of the connections of structural members can be caused by a combination of physical and/or chemical agents. The most common type of connections used for structural elements include bolting, riveting, pinning, and welding. These connections can fail through the overloading, fatiguing, or corrosion of the connectors. Common examples of this type of failure include the corrosion of bolt heads, rivets, and areas covered by fastening plates. The effective cross-sectional area of the connectors is often reduced by corrosion, making the connectors more susceptible to stress failure.

Typical metal frame structural problems are listed below.

Visible Defect	Possible Cause	Investigation
a. excessive deformations resulting from: out-of-plumb columns floors or supporting beams or column joints not at correct levels distorted or buckled members bowing of beams, ties and bracings lack of fit at connections damage to windows or cladding	poor fabrication or erection overloading wrong grade of steel, or size or stiffness of members inadequate bracing or lateral support poor fit, slip or failure of supports or connections torsional effects not accounted for in design unsatisfactory base or holding-down material or grouting impact; due to vehicles, forklift trucks, etc. elevated temperature effects not allowed for in design	structure may be unsafe, therefore assess conditions and give consideration to closure and temporary support check: original member design for under-size and/or lack of stiffness; existing load conditions against design and verify adequacy for load transference; effectiveness of connections, i.e. bolts, rivets or welds; line and level of beams and columns with original general arrangement; foundations for inadequacy or movement check whether structural arrangement is able to accommodate forces imposed by thermal movements.
b. unacceptable flexing of a complete structure or of individual members	effects from wind or rotating machinery not adequately allowed for in design vortex shedding on circular structures such as chimney (see also item above)	check on calm day or when machines are not working check if machinery is unbalanced or if anti-vibration mountings are deficient
c. member or members removed from a framework causing: partial failure of remaining structure; instability or overstressing of individual members; collapse	members removed by owner, maintenance engineers, etc. to provide opening or access	check effects of changed structural arrangement and assess if reinstatement of removed member is sufficient remedy

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Visible Defect	Possible Cause	Investigation
d. slight waviness or localized buckling or bulging generally at or near points of support or concentrated loads.	local yielding due to overload caused by high shear or buckling through an inadequate bearing length	check actual load, shear and bearing capacities against applied loads.
e. obvious absence of welds poor appearance of welds	oversight unsatisfactory welding	test for adequacy
f. hairlines or slight gaps on edges of material often revealed by flame cutting	laminations lamellar tearing	non-destructive testing
g. surface appearance of fractured material which may vary from smooth to crystalline cracks on surface of welds or heat-affected zones cracks initiated at stress points such as punched holes or sheared or flame-cut edges not subsequently machined or ground	poor welding fatigue caused by repetition of cyclic loading having reached the fatigue life of the structure or structural element brittle fracture, caused by low temperature combined with a tensile stress and notch effect, not allowed for in the design and selection of the steel	cause of metallurgical failure, indicated by the appearance of the fractured surface, is best determined by a welding specialist or metalurgist
h. slipping of joints and connections tearing or distortion of metal adjacent to holes	overloading incorrect type of fastener, e.g. undersize, wrong grade, etc. inadequately or over tightened bolts flame cut or poorly punched holes hole drifting during erection	check loading, size and effectiveness of fasteners

Visible Defect	Possible Cause	Investigation
i. open holes indicating missing fasteners rivets with loose or missing heads loose bolts, loose nuts (wrong size nuts) bent or distorted fasteners uneven seating of head or nut	fasteners badly fitted or never installed at time of original erection, through inadequate supervision or poor workmanship misaligned or misplaced holes failed fasteners because of overload or defective fastener material	check load adequacy and need to replace defective items
j. corrosion - varying between light rusting on previously protected surfaces to pronounced rusting on unprotected surfaces.	environmental attack presence of moisture failure of protective system	ascertain cause if moisture is present--Where is it coming from--can it be prevented? Is structural performance likely to be affected? Is sufficient sound material remaining to support the applied loading? note: Corrosion may be detrimental if present between surfaces of joints, due to the products of corrosion causing expansion

4.3 Methods of Test for Steel

Buildings generally are subjected to static loads, except for such conditions as seismic disturbances and vibrational loadings due to mechanical equipment. Consequently, flaws (such as cracks) in structural steel components of the building generally are not subject to progressive failure from fatigue, and it may not always be critical to evaluate them. If, however, it has been determined that a structural component (a weldment, splice plate, etc.) is suspect of degrading the structural integrity of the building, nondestructive evaluation methods can be extremely useful in evaluating the component. On the other hand, it is important to evaluate flaws other than cracks, such as corrosion, voids, pits, fabricating discontinuities, and porosity.

Most nondestructive evaluation methods offer more than just a superficial examination of the surface conditions of metals. It now is possible to reveal facts concerning overall properties, quality, and dimensions of both the surface and the internal regions of many metal test specimens.

Although nondestructive evaluation is used primarily on structural steel, some evaluation methods are applicable to other types of structural metals.

a. Visual Tests. For nondestructive examination of surface characteristics. Includes the use of borescopes, fiberoptics, panoramic cameras, and dye penetrants. Detects surface flaws, cracks, voids, holes, gouges, fabricating discontinuities, corrosion, pits, and other irregularities. Used to examine the surfaces of all metals. Permits examination of hidden surfaces (if access is available). Detects only defects visible to the eye. Limited to detection of surface flaws only.

b. X-ray or Gamma Ray. For detecting voids, porosity, inclusions, and cracks. Used on castings, forgings, weldments, and assemblies to check for fatigue, thickness gauging, internal flaws, etc. For all metals. Detects both internal and external flaws. Portable. Provides a permanent record on x-ray film. High cost. Heavy. Health hazard. Must have access to both sides of material.

c. Ultrasonic. For detecting cracks, voids, porosity, laps, segregated inclusions, poor brazing or bonding. (Will detect both surface and subsurface defects). Used for thickness gauging, material inspection of castings, forgings, and extrusions. For all metals. Locates small discontinuities; portable; instant results; accurate measure of thickness. Sensitivity is reduced by rough-surfaced parts. Odd-shaped pieces are hard to analyze. Requires skilled operator. Depends on operator's ability to interpret results and on orientation of the defect. Must couple transducer to surface of specimen carefully.

d. Magnetic Particle. For detecting cracks, seams, laps, voids, porosity, and inclusions. Used for surface and slightly subsurface inspection of parts sensitive to magnetization. Simple, inexpensive, senses flaws down to 1/4 inch below surface as well as surface flaws. Not applicable to non-magnetic metals or materials. It is messy. Careful surface preparation is required. Irrelevant indications often occur. Depends on the operator's ability to intepret results. Demagnetization after inspection may be necessary.

e. Coupon. For determining yield strength, yield point, tensile strength, elongation, modulus of elasticity, compressive strength. Used in die casting, forgings, structural shapes, malleable iron, powdered metals. Gives fast, accurate results of physical and mechanical values. Test is destructive since a sample must be removed to be tested.

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5
Reinforced Concrete Structures

5.1
General

The history of concrete construction is tied closely to improvements in cements, particularly the development of Portland cement in the 1870s; late 19th cen- tury improvements in material sciences and engineering, which rapidly increased confidence in the use of reinforced ("ferro-") concrete; and the advent of fireproof construction. Early 20th century industrial buildings in particular exploited the full potential of concrete construction, which has become common since World War II.

Problems with concrete are not new, but certainly they have become more visible in the last two decades. Concrete has been one of the most widely used artificial building materials of the last one hundred years. It has been applied as a structural material in reinforced frames or structural blocks, as a cladding material, and more recently as precast structural elements and panels. Now the material has reached a critical stage in its lifespan: a whole generation of reinforced concrete buildings and cast stone buildings built between 1900 and 1940 are beginning to require extensive repairs.

5.2 Structural Problems of Reinforced Concrete

Indications of deterioration and damage in reinforced concrete structures are usually limited and may have different implications. Cracking observed in the concrete, for example, may be the result of corrosion of the reinforcement, deflection, settlement, thermal expansion, contraction, or curing stresses. The location, configuration, and pattern of these cracks will be of significance. Spalling in general will be the result of corrosion of the reinforcement. Where staining has occurred, the color and location of the stains may be indicative; most often, brown staining will be the result of corrosion, but occasionally it may be caused by the aggregate. Disintegration of the surface of the concrete can be the result of salt crystallization, freeze-thaw cycles (often due to the use of deicing chemicals), and weathering.

Concrete patching procedures will usually suffice when damage is not extensive. Large amounts of corrosion and spalling, however, may require a detailed analysis of remaining structural capacity. The type and extent of repairs will be determined by the results of such analysis.

Precast members may present special problems. End supports should be checked for adequacy of bearing, indications of end shear problems, and restraint conditions.

Floor and roof systems of poured-in-place concrete with self-centering reinforcing, such as paper-backed mesh and rib lath, should be inspected for corrosion of the unprotected reinforcing.

Two publications by the American Concrete Institute (ACI) are especially useful for analyzing existing concrete structures: ACI 201 R-68, Guide for Making Condition Survey of Concrete in Service, and ACI 437 R-82, Strength Evaluation of Existing Concrete Buildings.

Typical reinforced concrete structural problems are listed below.

REINFORCED CONCRETE BEAMS AND SLABS

Visible Defect	Possible Cause	Investigation
a. vertical or slightly inclined cracks	overloading under-reinforcement, inadequate depth fault in design thermal movement shrinkage around stirrups	ascertain actual load being carried compared with design load compare span/depth ratio with design criteria check for uneven temperature gradient
b. diagonally inclined hair cracks generally at or near support of beams	overloading in shear thermal effects	ascertain shear capacity and compare with shear forces compare span/depth ratio with design criteria check for uneven temperature gradient
c. presence of diagonal cracks in beams on faces of and extending around the perimeter of section	torsional shear stresses	adequacy of design cause and magnitude of torsional moment
d. excessive deflection (damage to partitions/ glazing below in slabs and beams	inadequate depth overloading (long-term) fault in construction: tensile reinforcement out of position fault in design deterioration of materials	compare span/depth ratio with design criteria ascertain calculated theoretical deflections check loading history test concrete faulty design
e. cracking of slab and/or finishes over supports of slabs	slabs designed simply supported although continuous steel at supports inadequate or incorrectly positioned excessive relaxation of support moments in original design inadequate slab thickness	check for presence and location of top steel examination of design

REINFORCED CONCRETE GENERAL, INCLUDING COLUMNS, WALLS, AND CORBELS

Visible Defect	Possible Cause	Investigation
a. cracking of concrete cover/exposure of reinforcement very fine cracks are inherent in reinforced concrete rust staining on concrete surface	corrosion of reinforcement (e.g. by $CaCl_2$) nails and wire ties left in formwork corrosion of tying wires or chairs, etc. presence of iron containing compounds in the aggregate at surfaces inadequate cover fire	adequacy of concrete cover to reinforcement having regard to quality of concrete and severity of exposure visual examination--concrete usually white, straw or pink strength tests test for carbonization (phenolphthalein) analysis of concrete samples
b. fine hair crazing of surface	construction fault	
c. vertical fractures at intervals along rc walls	shrinkage moisture movement	provision or omission of movement joints amount and spacing of distribution reinforcement
d. diagonal fractures in rc walls	differential foundation or support settlement	adequacy of foundation on stiffness of support presence of water--ground movement
e. spalling of concrete from face or top of corbels	corrosion of reinforcement inaccurate positioning of top reinforcement inadequacy of top reinforcement lack of anchorage of top reinforcement overloading expansion of infilling brick panels elastic shortening and shrinkage of building frame frost attack shrinkage	adequacy of concrete cover in relation to severity of exposure pachometer checks design check examination of detail drawings (where possible) loading assessment and comparison with loading capacity provisions (or otherwise) of compression joints in panel walls
f. diagonal spalling front edge of nib	presence of horizontal and vertical forces on nib	thermal effects other horizontal effects

POST-TENSIONED EXTERNALLY OR INTERNALLY STRESSED CONCRETE BEAMS

Visible Defect	Possible Cause	Investigation
a. cracking or spalling of precast concrete with or without rust staining	corrosion of secondary reinforcement	check secondary reinforcement at cracked locations check cover, carbonation and chloride content of concrete if corrosion is confirmed
b. rust staining below mortar covering of externally stressed beams	corrosion of tendons	ascertain location of corrosion and dampness
c. excessive deflections anchorage loose or fractured separation at joints in beams of segmental construction tendons visible disturbance of mortar covering tendons of externally-stressed beams shear cracking on diagonals of beam segments	fracture of tendons overloading inadequate prestress	structure may be unsafe; examine and consider closure and propping investigate condition of the tendons if tendon fracture obvious check design against existing loading condition

PRETENSIONED CONCRETE BEAMS OR COLUMNS

Visible Defect	Possible Cause	Investigation
a. transverse cracking of concrete or excessive deflection	overloading inaequate pretension	check design against existing loading conditions
b. cracking or spalling of concrete with or without rust staining (cracking generally parallel to the direction of the steel)	corrosion of tendons or secondary steel reinforcement	check condition of exposed steel check cover, carbonation, and chloride content of concrete if corrosion is confirmed
c. bowing of columns	distortion during erection creep and shrinkage of concrete fracture of tendon if concrete cracked longitudinally	check conditions of load check adequacy of design check tendon condition at exposed locations

REINFORCED CONCRETE EXTERNAL WALLS AND SLAB BELOW GROUND

Visible Defect	Possible Cause	Investigation
a. random diagonal cracking or lateral cracking across bays at near-even spacing	inadequate provision for contraction/ shrinkage over-rich concrete mix	check amount of reinforcement and spacing of joints analysis of concrete samples
b. breaking up or spalling or surfaces	excessive wear due to traffic (forklift trucks, steel-tired vehicles, impact from machinery poor quality concrete chemical attack due to spillage frost attack	ascertain plant manufacturer's loadings test and analyze concrete check use--history

Visible Defect	Possible Cause	Investigation
c. wet and damp areas and deterioration of applied finishes	random cracking due to shrinkage of concrete inadequate expansion joints honeycombed or badly compacted concrete faulty filling of movement joints bad mix design inadequate reinforcement	examine spacing and detailing of joints pachometer test and analyze concrete
d. repetitive vertical fractures local settlements combined with diagonal cracks	excessive spacing of movement joints shrinkage poor compaction of subgrade/inadequate reinforcement ground movement, erosion, mining, or other outside causes	examine details removal of small areas to investigate conditions below slab

STONE OR PRECAST CLADDING

Visible Defect	Possible Cause	Investigation
a. spalling of vertical joints between stones and/or outward bowing of stonework in the horizontal direction	thermal movement	provision (or otherwise) of expansion joints and their adequacy adequacy of restraint fixing adequacy of stonework behind fixing adequacy of support for stonework thickness, strength and durability of stonework used conditions of fixing cramps used and suitability of material for purpose (i.e., ferrous, non-ferrous) evidence of stress corrosion (high tensile stress)

Visible Defect	Possible Cause	Investigation
b. spalling or horizontal joints between stones and/ or outward 'bowing' of stonework in the vertical direction	creep, shrinkage, and elastic shortening of building frame thermal movement	as above provision (or otherwise) of soft 'compression' joints between stones at story-height intervals and their adequacy
c. differential 'face' movement of one stone with another	fault fastening frost movement	failure or omission of fastenings condition of joints--likelihood of water penetration, lack of drainage in 'cavity' behind stone
d. vertical fractures in stonework	corroded structural steelwork or reinforcement in wall behind cladding shrinkage thermal movement	removal of one or more stones to ascertain cause
e. rust stains on surface, random occurrence	if unreinforced, likely to be iron bearing aggregate	if a problem, analyze concrete
f. rust stains on surface of precast concrete units suggest a pattern	corrosion of reinforcement possible use of calcium chloride in manufacture	check cover, carbonization, and analyze

5.3
Methods of Test for Concrete

Selection of the most appropriate and effective method for nondestructive evaluation of the concrete requires sound judgment based on the information required and the cost of the evaluation. Generally, on-site nondestructive or destructive testing should be preceded by a visual inspection of the concrete in all accessible parts of the structure, and is used to further evaluate the effects of already observed distress on the concrete members. A good general procedure to follow (in order of priority) is to use: 1) visual evaluation, 2) non-destructive evaluation, 3) destructive evaluation, and 4) load tests. In situ testing should be used primarily to verify strength and condition estimates already derived from the visual inspection.

a. Visual/Optical. For detection of surface flaws. May detect cracks 2-3 microns wide on a smooth surface (smaller with a hand-held magnifier). Used on concrete samples or in situ concrete structural components. Inexpensive because no special or power equipment is needed. Can yield defects not detectable by other methods. Only surface information is given. Relationship between surface appearance and condition of total sample must be determined.

b. Survey of Horizontal and Vertical Movement. For measuring differential movements over time. Structural movement may be detected within the precision of the surveying equipment. Used for long-term observations to determine critical movements of concrete structures. Cyclical relationships between deformation and temperature or load can be derived. Immediate data interpretations cannot be made because of the long-term, cyclical observation periods. A trained surveyor is necessary for data collection and evaluation.

c. Joint Survey. For checking a variety of joint conditions which may indicate problems with concrete, such as spalling, faulty joint fillers, seepage, and chemical attack. Depends upon the manual measuring device used (calipers, etc.). Measures expansion, contraction, or construction joints in concrete. An initial step of a more in-depth investigation of concrete problems.

Inexpensive because survey is limited to visual inspection and manual measurements. This method is most applicable to foundation walls and slabs. A trained evaluator is necessary for data collection and evaluation.

d. Fiber Optics. For detecting internal cracks, voids, or flaws if path to surface exists. Working length of 30 to 1330 mm (1-52 inches), depending on equipment used. Can be used to look into cracks or areas where cores have been removed. Clear, high-resolution image of remote inspection subjects. Many bore holes are required to give adequate success. Expensive.

e. Schmidt Rebound Hammer. For measuring surface hardness. Useful for determining strength of concrete. Used in concrete samples or in situ concrete structural members. Inexpensive, fast, and can be operated by laymen. The indication of concrete strength is not accurate and results are affected by condition of concrete surface.

f. Windsor Probe. For measuring the depth of penetration, which may correlate with concrete strength. Depends upon the location of the test and the precision of the calibrated depth gauge. Used on concrete samples or in situ concrete structural components. Equipment is simple, durable, and can be used by field personnel with little training. Slightly damages a small area of concrete that is 25 to 50 mm (1-2 inches) in diameter. Does not provide accurate determination of strength without a curve showing correlation between depth of penetration and concrete strength.

g. Magnetic. Used to locate steel reinforcement in concrete. Can detect ferromagnetic components only within 180 mm (7 inches) of concrete surface. Used on concrete samples or in situ concrete structural components. Light, portable equipment. Easy to operate and relatively inexpensive. Portable battery equipment will not operate satisfactorily in freezing temperatures. Good results are obtained only if one layer of rebars is present. Will not work well with mesh reinforcement.

h. High Energy Ultrasonics. For evaluating the depth quality and uniformity of concrete by measuring the velocity of high energy ultrasonic pulses. Thickness measurements of concrete accurate to ± 5.0 percent. For concrete samples or in situ concrete structural components. Measurements are very accurate. Currently, it is the only method to measure slab thickness accurately and nondestructively. Large and heavy power supply equipment is required; data interpretations are limited to thickness measurements. Both surfaces of the concrete must be accessible.

i. Radiographics. For determining density and internal structure of concrete, the location of rebars, and banding stress points. X-ray equipment can be used only on concrete laboratory samples; gamma ray equipment can be applied to concrete samples or in situ concrete structural components. Both provide a permanent record of problems on film. Very heavy and expensive for field use with concrete. Both radiation sources are

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injurious to living tissue and operators must be adequately shielded. Both surfaces of the concrete must be accessible.

j. <u>Microwave Absorption</u>. For determining moisture content and rebar size and location. Yields values of moisture content within 30 percent of the mean value. Used on concrete samples or in situ concrete structural components. Easy to use, and is moderately priced. Low degree of accuracy, and two opposite faces of specimen must be accessible.

k. <u>Ultrasonic Pulse Velocity, Resonant Frequency</u>. For determining concrete strength and quality. The estimations of uniformity and continuity are very qualitative in nature. Used on concrete samples or in situ concrete structural components. Excellent for determining concrete uniformity. Skill is required to analyze results. Does not provide an estimate of strength. Equipment is expensive and requires field calibration. Background vibrations can affect results.

l. <u>Load Testing</u>. For verifying load-carrying ability. Applied to in situ concrete structural systems. Provides a high degree of reliability on a structure's ability to perform under normal loading. Validity for long range performance is questionable. May cause cracks, distortion, or even premature failure. Also requires large amounts of preparation and clean-up time. Load tests should be performed in accordance with ACI 318-77.

m. <u>Radar</u>. For detection of substratum voids. Eighty percent reliability of void detection. Used for concrete samples or in situ concrete structural components. Far less destructive than "guess and drill" methods, and scanning of large surface areas can be done quickly.

Not reliable with slabs containing reinforcing mesh; very expensive; operator needs technical training.

n. Pullout Test. For measuring in situ strength of hardened concrete. Comparable to pull-out of cast-in-place anchors (ASTM C 900). Used on in situ concrete structural components. Fast, simple, inexpensive. Easy to apply in the field. Offers direct determination of strength parameters.

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Appendix 1
History of Structural Safety Code Provisions *

Turn-of-the-century building codes generally provided for structural safety for conventional construction through prescriptive requirements based on a body of experience of structural failures. "New" materials that required the use of structural analysis, such as iron, steel, and concrete were regulated by factors of safety criteria.

Many of the structural provisions of these early codes were copied or adapted from New York City's code. The National Building Code, an early "model" code published by the National Board of Fire Underwriters in 1905, and the Model Building Code, published in 1913 by the American School of Correspondence in Chicago, had structural provisions similar to New York's.

Often the structural provisions were incorrectly copied. An engineer from the American Bridge Company compared wind requirements of codes around the country and found significant variation in forces and allowable stresses. He wrote, "It might seem that our American municipalities have exhausted the combination of wind pressure and wind stresses that can be made." He cited one example where the Chicago, San Francisco, and Akron codes all specified a 20 psf wind force, but Chicago and San Francisco each permitted a 50 percent increase in allowable stress while Akron permitted no increase at all.

A brief synopsis of historical changes in structural design requirements is as follows:

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* Excerpted from Existing Buildings and Building Regulations, by Building Technology, Inc. for the U.S. Department of Housing and Urban Development, 1981.

(a) Vertical Loads

Floor Loads. Typically, building codes specify floor live load based on the type of occupancy. Floor loads specified in the New York City code of 1891 varied from 75 to 120 pounds per square foot (psf) for different occupancies. The 1901 New York City code revised these values to 60 psf for dwellings to 150 psf for storage occupancies. Current loads permitted by the New York City code are 40 psf for dwellings and 100-150 psf for storage occupancies, which are consistent with other recognized standards.

Roof Loads. The 1891 New York City code did not have a roof design criterion, but the 1901 code required a 50 psf load for flat roofs. The current New York City code calls for a 30 psf load for a similar roof. Roof loads are usually adjusted locally in areas of heavy snow and as such will show little consistency whether or not they are accurate.

(b) Wind Loads

The 1891 New York City code was silent on wind loads, but the 1901 code specified a design force of 30 psf. In addition, it provided that the overturning moment not exceed 75 percent of the righting dead-load moment. It also provided that buildings with obvious stability (4:1 height to width ratio) need not be designed to resist wind forces. The basic wind criteria have not changed significantly over the years except to increase the wind force at higher elevations.

(c) Seismic Loads

Early designers felt that seismic safety could be provided for in a manner similar to wind design. The inertial nature of seismic forces was not generally recognized until the early 1920's.

The 1927 Uniform Building Code included optional seismic design requirements based on building mass and soil type. These requirements were adopted almost unchanged in the California Administrative Code after the 1933 Long Beach earthquake, and did not change significantly until 1943 when Los Angeles developed a new formula that considered building height. San Francisco adopted similar requirements in 1947.

Seismic design requirements have continued to become more detailed in response to earthquake experiences both in the United States and abroad. The 1964 Alaska earthquake, the 1967 Caracas earthquake, and the 1971 San Fernando earthquake each resulted in code changes. Engineers today are generally of the opinion that the design values are realistic, and attention is now being directed to "non-structural" damage such as ceilings, partitions, curtain walls, and pipes.

(d) <u>Foundation (Geotechnic) Requirements</u>

The 1901 New York City code specified the following allowable foundation pressures: soft clay at 1 ton per square foot (TSF); wet clay and sand at 2 TSF; and hard stiff gravel at 4 TSF. Greater values were allowed for bedrock. These varied from 6 to 10 TSF.

Current codes require that soil bearing pressures be based on geotechnical reports. The values for clay and sand are about 50 percent of those allowed at the turn-of-the-century. This reduction in soil bearing value is in response to settlement and other foundation problems which, while not producing catastrophic failure, can be the source of many building problems such as wall cracking and pipe breakage.

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(e) Local Strength

Only prescriptive requirements for masonry walls were included in the early codes. These requirements were based on several bracing factors, including the unsupported height of the wall, the distance between cross walls or buttresses, and the span of floor joists. Exterior walls and bearing walls required greater thickness (probably for both wind and fire resistance) than was required for interior non-bearing walls.

(f) Materials Criteria

Older codes generally granted the building official broad authority to accept or reject materials. Cast iron, for example, called for consistent gray color and an inspection hole to permit the inspector to verify the thickness of the column walls.

(g) Factors of Safety and Allowable Stresses

Early codes specified factors of safety for building elements in bending (beams), compression (columns) and tension (trusses). This factor of safety was to be used with the ultimate strength of materials specified by Trautwine in The Civil Engineers Pocketbook of 1871 and later editions (published through the early 1930s), and designed in accordance with the principles of mechanics published by the U.S. Military Academy at West Point, or the works of Kidder and others. The 1891 New York City code specified a factor of safety of 3 for beams, 6 for columns, and 6 for tensile members. These factors were to be used with the ultimate strength of materials as noted by Trautwine and others. By 1901 the New York City code included actual working stresses. Materials listed were cast and wrought iron, steel, and wood. The stresses permitted in the 1901 code remained essentially unchanged for the next 20 years.

The 1901 New York City code also included some innovations including allowable stress increases when combining wind forces and dead and live loads. A stress increase of 50 percent was allowed. The code also permitted a reduction of live loads in footings and columns. The reduction was 5 percent per story with a 50 percent maximum.

In the 1920s under the direction of then Secretary of Commerce Herbert Hoover, the U.S. Bureau of Standards developed a series of building code recommendations. These were included in the Building and Housing Series (BH). This series had a profound effect on the structural provisions of building codes. Publications of importance to structural design were:

BH 1 - Small Dwelling Construction, 1924

BH 6 - Recommend Minimum Requirements for Masonry Wall Construction, 1924

BH 7 - Minimum Live Load Allowed for Use in Design of Buildings, 1925

BH 9 - Recommended Building Code Requirements for Working Stress in Building Materials, 1926

H 18 - Recommended Minimum Requirements for Small Dwelling Construction, 1932.

These documents, developed by consensus committee, provided the format and basic technical content and structural provisions that are still used in the codes today. Since the 1920s, there have been significant advances in the art of structural analysis. Two worthy of historical note are Hardy Cross's work, published in 1932, on moment analysis of building frames, and Terzaghi and Peck's work, published in 1948, on specialized analytical techniques. These and related structural engineering advances have continued to shape and refine the code requirements for more complex building types.

and normally 300^{o}C may be taken as the transition temperature. For the standard fire and various periods of heating, the following show results for slabs:

Heating period	Maximum surface temperature		Maximum depth of concrete showing characteristic change mm			
hours	^{o}F	^{o}C	Pink or red 300^{o}C	fading of red, friability 600^{o}C	buff 950^{o}C	sintering 1200^{o}C
1	1742	950	56	19	0	0
2	1922	1050	100	38	6	0
4	2246	1230	140	63	25	3
6	2282	1250	170	90	38	6

The temperature within a slab may continue to rise after the fire is ended, and some of the above maxima were attained after the end of the heating period.

Analysis of the damage and assessment of the necessary repairs may be possible within a reasonable degree of accuracy, but final acceptance may depend on proof by load test. Performance is generally judged in terms of recovery of deflection after removal of the load.

1. TIMBER

Timber 'browns' at about 120^{o}- 150^{o}C, 'blackens' around 200^{o}-250^{o}C, and emits combustible vapors at about 300^{o}C. Above a temperature of 400^{o}-450^{o}C (or 300^{o}C if a flame is present) the surface of the timber will ignite and char at a steady rate. Any charred part of a section must be assumed to have lost all strength, but timber beneath the charred layer may be assumed to have no significant loss of strength because the thermal conductivity of timber is low. The table below gives rate of charring.

thermal expansion of the two faces, and behavior is influenced by edge conditions. For solid bricks, resistance to the effects of fire is directly proportional to thickness. Perforated bricks and hollow clay units are more sensitive to thermal shock: there can be cracking of the connecting webs and tendency for the wythes to separate. With cavity walls the inner wythe carries the major part of the load. Such exterior walls can be subjected to more severe forces by heated and expanding floor slabs than an internal wall. All types of brick give much better performance if plaster is applied, thus giving improved insulation and reducing thermal shock.

Analysis and Repair

It will be possible to determine from the color change of the mortar or of the bricks themselves the degree of heating of the wall. For solid brick walls without undue distortion, the portion beyond the pink or red boundary may be considered serviceable and calculations made accordingly. Perforated and hollow brick walls should be inspected carefully for the effects of thermal shock and may need replacement. Plastered bricks sometimes suffer little damage and may need replacement of the surface treatment only.

3. STEEL

For steel, the yield strength is reduced to about half at 550°, and at 1000°C it is 10 percent or less. Because of its high thermal conductivity, the temperature of unprotected internal steelwork will normally be little different from the fire temperature. Structural steelwork is therefore usually insulated.

Apart from losing practically all its loadbearing capacity, unprotected steelwork when sufficiently heated can undergo considerable expansion, the coefficient being on the order 10^{-5} per $^{\circ}$C. Young's modulus does not decrease with temperature so rapidly as yield strength.

Cold-worked reinforced bars, when heated, lose their strength more rapidly than hot-rolled high-yield bars and mild-steel bars. The properties after heating are of interest from the point of view of reinstatement. The original yield stress is almost completely recovered on cooling from a temperature of 500°-600°C for all bars, but on cooling from 800°C it is reduced by 30 percent for cold-worked bars and 5 percent for hot-rolled bars.

For prestressing steels, the loss of strength occurs at lower temperatures than for reinforcing bars. Cold-drawn and heat-treated steels lose a part of their strength permanently when heated to temperatures in excess of about 300°C and 400°C, respectively.

The creep rate of steel is very sensitive to higher temperatures; it becomes significant for mild steel above 450°C and for prestressing steel above 300°C. In fire resistance tests, the rate of temperature rise when the steel is reaching its critical temperature is fast enough to mask any effects of creep, but where there may be a long cooling-down period, as in prestressed concrete, subsequent creep may have some effect in an element that has not reached the critical condition.

Analysis and Repair

In general, a structural steel member remaining in place, with negligible or minor distortions to the web, flanges, or end connections should be considered satisfactory for further service. The exception will be for the relatively small number of structures in cold-worked or tempered steel where there may be permanent loss of strength. The change in strength may be assessed using estimates of the maximum temperatures attained or by on-site testing. If necessary, the steel should be replaced, although reinforcement with plates may be possible. Microscopy can be used to determine changes in microstructure. Since this is a specialized field, the services of a metallurgist are essential.

4. CONCRETE

For concrete, compressive strength varies not only with temperature but also with a number of other factors including the rate of heating, the duration of heating, whether the specimen was loaded or not, the type and size of aggregate, percentage of cement paste, and the water/cement ratio. In general, it may be said that concrete heated by a building fire always loses some compressive strength and continues to lose it on cooling. However, where the temperature has not exceeded 300°C most strength eventually will be recovered.

Because of the comparatively low thermal diffusivity of concrete (of the order of 1mm/s) the 300°C contour may be at only a small depth below the heated face. The modulus of elasticity also decreases with temperature, although it is believed that it will recover substantially with time, provided that the coefficient of thermal expansion of the concrete is of the order of 10^{-5} per °C (but this varies with aggregate). Creep becomes significant at quite low temperatures, being of the orders of 10^{-4} to 10^{-3} per hour over the temperature range 250°C to 700°C, and can have a beneficial effect in relaxing stresses.

Analysis and Repair

a. Effective cross-section. Removal of the surface material down to the red boundary (see table in Introduction, above) will reveal the remaining cross-section that can be deemed effective. Compression tests of cores can indicate the strength of this concrete, giving a value to be adopted to calculations.

b. Cracks. Most fine cracks are confined to the surface. Major cracks that could influence structural behavior are generally obvious. A wide crack or cracks near supports may mean there has been a loss of anchorage of the reinforcement.

c. Reinforcement. Provided that mild steel or hot-rolled high-yield steel is undistorted and has not reached a temperature above about 800°C, it may be assumed to have resumed its original properties, but cold-worked bars will have suffered some permanent loss.

d. Prestressing steel. It is likely that prestressing steel will have lost some strength, particularly if it has reached temperatures over 400°C. There will also be a loss of tensile stress. These effects can be assessed for the estimated maximum temperature attained.

e. Remedial measures. In some situations, replacement of a damaged member may be the most practical and economic solution. Elsewhere, repair of the member, even if extensive, will be justified in order to avoid inconvenience and loss or damage to other structural members.

Where new members are connected to existing ones, monolithic action must be ensured: this calls for careful preparation of the concrete surfaces and continuity of steel. For repair, the removal of all loose friable concrete is essential to ensure adequate bonding. Extra reinforcement should be fixed by experienced welders.

New concrete may be placed either by casting with formwork or by the gunite method; with the latter it may be possible to avoid increasing the original dimensions of the member. The choice of method will depend on the thickness of the new concrete, the surface finish required, the possibility of placing and compacting concrete in the formwork and the degree of importance attached to an increase in size of section.

Large cracks can be sealed by injection of a latex solution, resin, or by various epoxy preparations. Various washes or paints are available to restore the appearance of finely cracked or crazed surfaces.

Appendix 3
Wood Inhabiting Organisms *

Wood is a porous material and will absorb moisture from the air. Moisture is attracted to the walls of the tubes which make up the wood. As walls absorb moisture, the wood swells. If the humidity is kept at 100 percent, the walls become saturated with water. The moisture content at which this occurs is the fiber saturation point: approximately 30 percent by weight for most species used in construction. Fungi will only decay wood with a moisture content above the fiber saturation point. To allow a safety margin, wood with a moisture content above 20 percent is considered to be susceptible to decay. Wood in properly constructed buildings seldom will have a moisture content above 16 to 18 percent. Thus, wood will only decay if it is in contact with the ground or wetted by an external source of moisture such as rain seepage, plumbing leaks, or condensation. Dry wood will never decay. Also, the drier the wood, the less likely it is to be attacked by most types of wood-inhabiting insects.

Wood-inhabiting fungi are small plants which lack chlorophyll and use wood as their food source. Some fungi use only starch and proteins in the wood and don't weaken it. Others use the structural components, and as they grow they weaken the wood, which eventually becomes structurally useless. All fungi require moisture, oxygen, warmth, and food. The keys to preventing or controlling growth of fungi in wood in buildings are to either keep wood dry (below a moisture content of 20 percent) or to

* Excerpted from A Guide to the Inspection of Existing Homes for Wood Inhabiting Insects, by Michael P. Levy for the U.S. Department of Housing and Urban Development (undated).

use preservative- treated or naturally-resistant heartwood or selected species.

Wood-inhabiting insects can be divided into those which use wood as a food material--termites and wood-boring beetles, for example, and those which use it for shelter--carpenter ants and bees, for example. Termite damage is caused by immature forms called nymphs; for wood-boring beetles, by larvae or grubs; and for ants and bees, by adult insects.

Some wood-inhabiting organisms are found in all parts of the country. Others are highly localized. Some, although common, cause very little structural damage. The following is a description of the major wood-inhabiting fungi and insects in the United States.

a. Surface Molds and Sapstain Fungi. Surface molds or mildew fungi discolor the surface of wood, but do not weaken it. They are generally green, black, or orange, and powdery in appearance. The various building codes allow the use of framing lumber with surface molds or mildew, providing that the wood is dry and not decayed. Spores (or seeds) of surface molds or mildew fungi grow quickly on moist wood, or on wood in very humid conditions. They can grow on wood before it is seasoned, when it is in the supplier's yard or on the building site, or in a finished house. When the wood dries, the fungi die or become dormant, but they do not change their appearance. Thus, wherever surface molds or mildew fungi are observed on wood in a building it is a warning sign that at some time the wood was moist, or humidity was high.

Surface molds and mildew fungi are controlled by eliminating the source of high humidity or excess moisture, for example by repairing leaks, improving ventilation in attics or crawl spaces, or installing soil covers. Before taking corrective action, the source of moisture which allowed fungus growth must be determined. If the wood is dry and the sources of moisture are no longer present, no corrective action need be taken.

Sapstain or bluestain fungi are similar to surface molds, except that discoloration goes deep into the wood. They color the wood blue, black, or gray and do not weaken it. They grow quickly on moist wood and do not change their appearance when they die or become dormant. They usually occur in the living tree, or before the wood is seasoned, but sometimes in the supplier's yard, on the building site, or in a finished house. In the latter case they are normally associated with rain seepage or leaks. Stain fungi are a warning sign that at some time the wood was moist. Control is the same as for surface molds or mildew fungi.

b. <u>Water-conducting Fungi</u>. Most decay fungi are able to grow only on moist wood and cannot attack adjacent dry wood. Two brown-rot fungi, Poria incrassata and Merulius lacrymans, are able to conduct water for several feet through root-like strands or rhizomorphs, moisten wood, and then decay it. These are sometimes called water-conducting or dry-rot fungi. They can decay wood in houses very rapidly, but fortunately they are quite rare. Poria incrassata is found most frequently in the Southeast and West. Merulius lacrymans occurs in the Northeast. Both fungi can cause extensive damage in floors and walls away from obvious sources of moisture. Decayed wood has the characteristics of brown rotted wood except that the surface of the wood sometimes appears wavy but apparently sound, although the interior may be heavily decayed. The rhizomorphs which characterize these fungi can be up to an inch in diameter and white to black in color, depending on their age. They can penetrate foundation walls, and often are hidden between wood members. The source of moisture supporting fungal growth must be found and eliminated to control decay. Common sources include water leaks and wood in contact with or close to the soil: for example, next to earth-filled porches or planters. Where the fungus grows from a porch, the soil should be removed from the porch next to the foundation wall to prevent continued growth of the fungus into the house. Poria incrassata normally occurs in new or remodeled houses and can cause extensive damage within 2 to 3 years.

c. Brown & White Rot. The fungi often produce a whitish, cottony growth on the surface of wood. They grow only on moist wood. The fungi can be present in the wood when it is brought into the house, or can grow from spores which are always present in the air and soil. Wood attacked by these fungi should not be used in construction.

Wood decayed by brown-rot fungi is brittle and darkened in color. As decay proceeds, the wood shrinks, twists, and cracks perpendicular to the grain. Finally, it becomes dry and powdery. Brown-rot is the commonest type of decay found in wood in houses.

Wood decayed by white-rot fungi is fibrous and spongy, and is bleached in color. Sometimes it has thin, dark lines around decayed areas. The wood does not shrink until decay is advanced.

The fungi can be controlled by eliminating the source of moisture which allows them to grow: for example by improving drainage and ventilation under a house, repairing water leaks, or preventing water seepage. When the wood dries, the fungi die or become dormant. Spraying wood with chemicals does not control decay. If the moisture source cannot be eliminated, all the decayed wood should be replaced with pressure- treated wood.

d. <u>White-pocket Rot</u>. White-pocket rot is caused by a fungus which attacks the heartwood of living trees. Decayed wood contains numerous small, spindle-shaped white pockets filled with fungus. These pockets are generally 1/8 to 1/2 inch long. When wood from infected trees is seasoned, the fungus dies. Therefore, no control is necessary. White-pocket rot generally is found in softwood lumber from the West Coast.

e. <u>Subterranean Termites</u>. Subterranean termites normally damage the interior of wood structures. Shelter tubes are the commonest sign of their presence. Other signs include structural weakness of wood members, shed wings or warmers, soil in cracks or crevices, and dark or blister- like areas on wood. The major characteristics of infested softwood when it is broken open are that damage is normally greatest in the softer springwood, and gallery walls and inner surfaces of shelter tubes have a pale, spotted appearance like dried oatmeal. The galleries often contain a mixture of soil and digested wood. Termites usually enter houses through wood in contact with the soil or by building shelter tubes on foundation walls, piers, chimneys, plumbing, weeds, etc. Although they normally maintain contact with the soil, subterranean termites can survive when they are isolated from the soil if they have a continuing source of moisture. Heavy damage by subterranean termites (except Formosans) does not normally occur within the first five to ten years of a building's life, although attack may start as soon as it is built. Subterranean termites can be controlled most effectively by the use of chemicals in the soil and foundation area of the house, by breaking wood-soil contact, and by eliminating excess moisture in the house. When applied properly, chemicals such as chlordane, heptachlor, aldrin, or dieldrin will prevent or control termite attack for at least 25 years.

f. <u>Formosan Subterranean Termites</u>. Formosan subterranean termites are a particular vigorous species of subterranean termite which has spread to this country from the Far East. They cause considerable damage in Hawaii and Guam and have been found in several locations on the United States mainland. It is anticipated that they could eventually become established along Southern Coasts, the lower East and West Coasts, in the lower Mississippi Valley, and in the Caribbean.

The most obvious characteristics which distinguish Formosan subterranean termite swarmers from those of native species are their larger size (up to 5/8 inch compared to 1/3 to 1/2 inch) and hairy wings (compared with smooth wings in other subterraneans). Soldiers have oval shaped heads, as opposed to the oblong and rectangular heads of native soldier. Formosan termites also produce a hard material called carton which resembles sponge. This is sometimes found in cavities under fixtures, or in walls adjacent to attacked wood. Other characteristics--and control methods--are similar to those for native subterranean termites. However, Formosan subterranean termites are more vigorous, and can cause extensive damage more rapidly than do native species. For this reason Formosans should be controlled as soon as possible after discovery.

g. <u>Drywood Termites</u>. It is quite common for buildings to be infested by drywood termites within the first five years of their construction in southern California, southern Arizona, southern Florida, the Pacific area, and the Caribbean. Swarmers generally enter through attic vents or shingle roofs, but in hot, dry locations they can be found in crawl spaces. Window sills and frames are other common entry points.

Drywood termites live in wood that is dry. They require no contact with the soil or with any other source of moisture. The first sign of drywood termite infestation is usually piles of fecal pellets, which are hard, less than 1/25 inch in length, with rounded ends and six flattened or depressed sides. The pellets vary in color from light gray to very dark brown, depending on the wood being consumed. The pellets, eliminated from galleries in the wood through round "kick holes," accumulate on surfaces or in spider webs below the "kick holes." There is very little external evidence of drywood termite attach in wood other than the pellets. The interior of damaged wood has broad pockets or chambers which are connected by tunnels that cut across the grain through spring and summerwood. The galleries are perfectly smooth and have few if any surface deposits. There are usually some fecal pellets stored in unused portions of the galleries. Swarming is another sign of termite presence.

It normally takes a very long time for the termites to cause serious weakness in house framing. Damage to furniture, trim, and hardwood floors can occur in a few years. The choice of control method depends on the extent of damage. If the infestation is

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widespread or inaccessible, the entire house should be fumigated. If infestation is limited, spot treatment can be used, or the damaged wood can be removed.

h. Dampwood Termites. Dampwood termites of the desert Southwest and southern Flordia are rarely of great danger to structures. Pacific Coast dampwood termites can cause damage greater than subterranean termites if environmental conditions are ideal.

Dampwood termites build their colonies in damp, sometimes decaying wood. Once established, some species extend their activities to sound wood. They do not require contact with the ground, but do require wood with a high moisture content. There is little external evidence of the presence of dampwood termites other than swarmers or shed wings. They usually are associated with decayed wood. The appearance of wood damaged by dampwood termites depends on the amount of decay present. In comparatively sound wood, galleries follow the springwood. In decayed wood, galleries are larger and pass through both spring and summerwood. Some are round in cross section, others oval. The surfaces of the galleries have a velvety appearance and are sometimes covered with dried fecal material. Fecal pellets are about 1/25 inch long and colored according to the kind of wood being eaten. Found throughout the workings, they are usually hard, and round at both ends. In very damp wood the pellets are often spherical or irregular, and may stick to the sides of the galleries.

Dampwood termites must maintain contact with damp wood. Therefore, they can be controlled by eliminating damp wood. Treatment of the soil with chemicals can also be used to advantage in some areas.

i. <u>Carpenter Ants</u>. Carpenter ants burrow into wood to make nests, and do not feed on the wood. They commonly nest in dead portions of standing trees, stumps, logs and sometimes wood in houses. Normally they do not cause extensive structural damage. Most species start their nests in moist wood that has begun to decay. They attack hardwoods and softwoods. The most obvious sign of infestation is the large reddish-brown to black ants, 1/4 to 1/2 inch long, inside the house. Damage occurs in the interior of the wood. There may be piles or scattered bits of wood powder (frass) which are very fibrous and sawdust- like. If the frass is from decayed wood, pieces tend to be darker and more square ended. The frass is expelled from cracks and crevices, or from slit-like openings made in the wood by the ants. It is often found in basements, dark closets, attics, under porches, and in crawl spaces. Galleries in the wood extend along the grain and around the annual rings. The softer springwood is removed first. The surfaces of the galleries are smooth, as if they had been sandpapered, and are clean. The most effective way to control carpenter ants is to locate the nest and kill the queen in colonies in and near the house with insecticides. It is sometimes also helpful to treat the voids in walls, etc. For current information on control, an entomologist should be contacted.

j. <u>Wood-boring Beetles, Bees, and Wasps</u>. There are numerous species of wood-boring insects which occur in houses. Some of these cause considerable damage if not controlled quickly. Others are of minor importance and attack only unseasoned wood. Beetles, bees, and wasps all have larval, or grub, stages in their life cycles, and the mature flying insects produce entry or exit holes in the surface of the wood. These holes, and sawdust from tunnels behind the holes, are generally the first evidence of attack visible to the building inspector. Correct identification of the insect responsible for the damage is essential if the appropriate control method is to be selected. The characteristics of each of the more common groups of beetles, bees, and wasps are discussed in the following table which summarizes the size and shape of entry or exit holes produced by wood-boring insects, the types of wood they attack, the appearance of frass or sawdust in insect tunnels, and the insect's ability to reinfest wood in a house.

To use the table, match the size and shape of the exit or entry holes in the wood to those listed in the table; note whether the damaged wood is a hardwood or softwood and whether damage is in a new or old wood product (evidence of inactive infestations of insects which attack only new wood will often be found in old wood; there is no need for control of these). Next, probe the wood to determine the appearance of the frass. It should then be possible to identify the insect type. It is clear from the table that there is often considerable variation within particular insect groups. Where the inspector is unsure of the identity of the insect causing damage, a qualified entomologist should be consulted.

k. Lyctid Powder-post Beetles. Lyctids attack only the sapwood of hardwoods with large pores: for example, oak, hickory, ash, walnut, pecan, and many tropical hardwoods. They reinfest seasoned wood until it disintegrates. Lyctids range from 1/8 to 1/4 inch in length and are reddish-brown to black. The presence of small piles of fine flourlike wood powder (frass) on or under the wood is the most obvious sign of infestation.

Shape and Size (inches) of Exit/Entry Hole	Wood Type	Age of Wood Attacked	Appearance of Frass in Tunnels	Insect Type	Reinfest
round 1/50-1/8	softwood & hardwood	new	none present	ambrosia beetles	no
round 1/32-1/16	hardwood	new & cld	fine, flour-like, loosely packed	lyctid beetles	yes
round 1/16-3/32	bark/sapwood interface	new	fine to course, bark colored, tightly packed	bark beetles	no
round 1/16-1/8	softwood & hardwood	new & old	fine powder and pellets, loosely packed; pellets may be absent and frass tightly packed in some hardwoods	anobiid beetles	yes
round 3/32-9/32	softwood & hardwood (bamboo)	new	fine to coarse powder, tightly packed	bostrichid beetles	rarely
round 1/6-1/4	softwood	new	coarse, tightly packed	horntail or woodwasp	no
round 1/2	softwood	new & old	none present	carpenter beetle	yes
round-oval 1/8-3/8	softwood & hardwood	new	coarse to fibrous, mostly absent	round-headed borer	no
oval 1/8-1/2	softwood & hardwood	new	sawdust-like, tightly packed	flat-headed borer	no
oval 1/4-3/8	softwood	new & old	very fine powder and tiny pellets, tightly packed	old house borer	yes
flat oval 1/2 or more or irregular surface groove 1/8-1/2 wide	softwood & hardwood	new	absent or sawdust-like, coarse to fibrous; tightly packed	round or flat headed borer	no

Even a slight jarring of the wood makes the frass sift from the holes. There are no pellets. The exit holes are round and vary from 1/32 to 1/16 inch in diameter. Most of the tunnels are about 1/16 inch in diameter and loosely packed with fine frass. If damage is severe, the sapwood may be completely converted within a few years to frass held in by a very thin veneer of surface wood with beetle exit holes. The amount of damage depends on the level of starch in the wood. Infestations are normally limited to hardwood paneling, trim, furniture, and flooring. Replacement or removal and fumigation of infested materials are usually the most economical and effective control methods. For current information on the use of residual insecticides, the inspector should contact the extension entomologist at his nearest land grant university, or a reputable pest control company.

1. <u>Anobiid Beetles</u>. The most common anobiids attack the sapwood of hardwoods and softwoods. They reinfest seasoned wood if environmental conditions are favorable. Attacks often start in poorly heated or ventilated crawl spaces and spread to other parts of the house. They rarely occur in houses on slab foundations. Anobiids range from 1/8 to 1/4 inch in length and are reddish-brown to nearly black. Adult insects are rarely seen. The most obvious sign of infestation is the accumulation of powdery frass and tiny pellets underneath infested wood or streaming from exit holes. The exit holes are round and vary from 1/16 to 1/8 inch in diameter. If there are large numbers of holes and the powder is bright and light-colored like freshly sawed wood, the infestation is both old and active. If all the frass is yellowed and partially caked on the surface where it lies, the infestation has been controlled or has died out

naturally. Anobiid tunnels are normally loosely packed with frass and pellets. It is normally 10 or more years before the numbers of beetles infesting wood become large enough for their presence to be noted. Control can be achieved by both chemical and non-chemical methods. For current information on control of anobiids, the inspector should contact the extension entomologist at his nearest land grant university, or a reputable pest control company.

m. Bostrichid Powderpost Beetles. Most bostrichids attack hardwoods, but a few species attack softwoods. They rarely attack and reinfest seasoned wood. Bostrichids range from 1/8 to 1/4 inch in length and from reddish- brown to black. The black polycaon is an atypical bostrichid and can be 1/2 to 1 inch in length. The first signs of infestation are circular entry holes for the egg tunnels made by the females. The exit holes made by adults are similar, but are usually filled with frass. The frass is meal-like and contains no pellets. It is tightly packed in the tunnels and does not sift out of the wood easily. The exit holes are round and vary from 3/32 to 9/32 inch in diameter. Bostrichid tunnels are round and range from 1/16 to 3/8 inch in diameter. If damage is extreme, the sapwood may be completely consumed. Bostrichids rarely cause significant damage in framing lumber and primarily affect individual pieces of hardwood flooring or trim. Replacement of structurally weakened members is usually the most economical and effective control method.

n. Old House Borer sapwood of softwoods, primarily pine. It reinfests seasoned wood, unless it is very dry. The old house borer probably ranks next to termites in the frequency with which it occurs in houses in the Middle Atlantic States. The beetle ranges from 5/8 to 1 inch in length, and is brownish-black in color. The first noticeable sign of infestation by the old house borer may be the sound of larvae boring in the wood. They make a rhythmic ticking or rasping sound, much like a mouse gnawing. In severe infestations the frass, which is packed loosely in tunnels, may cause the thin surface layer of the wood to bulge out, giving the wood a blistered look. When adults emerge (3 to 5 years in the South; 5 to 7 years in the North), small piles of frass may appear beneath or on top of infested wood. The exit holes are oval and 1/4 to 3/8 inch in diameter. They may be made through hardwood, plywood, wood siding, trim, sheetrock, paneling, or flooring. The frass is composed of very fine powder and tiny blunt-ended pellets. If damge is extreme, the sapwood may be completely reduced to powdery frass with a very thin layer of surface wood. The surfaces of the tunnels have a characteristic rippled pattern like sand over which water has washed. Control can be achieved by both chemical and non-chemical methods. For current information on control of the old house borer, the inspector should contact the extension entomologist at his nearest land grant university or a reputable pest control company.

o. Carpenter Bees. Carpenter bees usually attack soft and easy-to-work woods, such as California redwood, cypress, cedar, and Douglas fir. Bare wood, for example, unfinished siding or roof trim is preferred. The only external evidence of attack is the entry holes made by the female. These are round and 1/3 inch in diameter. A rather course sawdust-like frass may accumulate on surfaces below the entry hole. The frass is usually the color of freshly sawed wood. The presence of carpenter bees in wood sometimes attracts woodpeckers, which increases the damage to the surface of the wood. The carpenter bee tunnels turn at a right angle after extending approximately an inch across the grain of the wood, except when entry is through the end of a board. They then follow the grain of the wood in a straight line, sometimes for several feet. The tunnels are smooth-walled. It takes several years of neglect for serious structural failure to occur. However, damaged wood is very unsightly, particularly if woodpeckers have followed the bees. The bees can be controlled by applying 5 to 10 percent carbaryl (Sevin) dust into the entry holes. Several days after treatment, the holes should be plugged with dowel or plastic wood. Prevention is best achieved by painting all exposed wood surfaces.

p. Other Wood-inhabiting Insects. There are several other species of insects which infest dying or freshly felled trees or unseasoned wood, but which do not reinfest seasoned wood. They may emerge from wood in a finished house, or evidence of their presence may be observed. On rare occasions, control measures may be justified to prevent disfigurement of wood, but control is not needed to prevent structural weakening.

Ambrosia Beetles. These insects attack unseasoned sapwood and heartwood of softwood and hardwood logs, producing circular bore holes 1/50 to 1/8 inch in diameter. Bore holes do not contain frass, but are frequently stained blue, black, or brown. The insects do not infest seasoned wood.

Bark Beetles. These tunnel at the wood/bark interface and etch the surface of wood immediately below the bark. Beetles left under bark edges on lumber may survive for a year or more as the wood dries. Some brown, gritty frass may fall from circular bore holes in the bar, diameter 1/16 to 3/32 inch. These insects do not infest wood.

Horntails (Wood Wasps). Horntails generally attack unseasoned softwoods and do not reinfest seasoned wood. One species sometimes emerges in houses from hardwood firewood. Horntails occasionally emerge through paneling, siding, or sheetrock in new houses; it may take 4 to 5 years for them to emerge. They attack both sapwood and heartwood, producing a tunnel which is roughly C-shaped in the tree. Exit holes and tunnels are circular in cross section, with diameter 1/6 to 1/4 inch. Tunnels are tightly packed with course frass. Frequently, tunnels are exposed on the surface of lumber by milling after development of the insect.

Round-headed Borers. Several species are included in this group. They attack sapwood of softwoods and hardwoods during storage, but rarely attack seasoned wood. The old house borer is the major round-headed borer which can reinfest seasoned wood. When round-headed borers emerge from wood, they make slightly oval to nearly round exit holes 1/8 to 3/8 inch in diameter. Frass varies from rather fine and meal- like in some species to very course fibers like pipe tobacco in others. Frass may be absent from tunnels, particularly where the wood was machined after emergence of the insects.

Flat-headed Borers. These borers attack sapwood and heartwood of softwoods and hardwoods. Exit holes are oval, with the long diameter 1/8 to 1/2 inch. Wood damaged by flat-headed borers is generally sawed after damage has occurred, so tunnels are exposed on the surface of infested wood. Tunnels are packed with sawdust-like borings and pellets, and tunnel walls are covered with fine transverse lines somewhat similar to some round-headed borers. However, the tunnels are much more flattened. The golden buprestid is one species of flat-headed borer which occurs occasionally in the Rocky Mountain and Pacific Coast States. It produces an oval exit hole 3/16 to 1/4 inch across, and may not emerge from wood in houses for 10 or more years after infestation of the wood. It does not reinfest seasoned wood.

If signs of insect or fungus damage other than those already described are observed, the inspector should have the organism responsible identified before recommending corrective measures. Small samples of damaged wood, with any frass and insect specimens (larvae or grubs must be stored in vials filled with alcohol), should be sent for identification to the entomology or pathology department of the state land grant university.

Appendix 4
Laws Governing the Inspection and Repair of Building Facades *

Laws passed in several major cities over the past decade require that buildings taller than a specified height be inspected periodically to protect the public from facade failures. The requirements of two of these laws--from the cities of Chicago and New York are outlined below.

<u>New York City</u>

New York's law became effective in 1980. It requires periodic inspections by an architect or engineer of all exteriors on structures exceeding six stories in height. Following such inspections, detailed reports must be filed with the New York City Department of Buildings. The law was

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from both the public and private sectors. The subcommittee assisted in the preparation and enactment of the first ordinance in the United States covering facade failures. This was followed by the establishment of guidelines which became a part of the ordinance. The law was enacted in 1978. The guidelines were designed to establish a uniform standard for the facade examinations and written reports.

The Chicago statute called for an examination of all buildings over five stories in height. Inspection reports had to be completed and filed within two years of passage.

Inspection of new construction must be completed within the fifth year after completion of the building. Periodic inspections are required at least once every ten years for buildings less than 35 years old, and once every five years for buildings 35 years or older.

The law calls for a critical examination to be undertaken followed by a written report prepared under the direct supervision of a registered architect or structural engineer. The report must "clearly document" existing conditions of all walls and enclosures and include a record of all significant observable deterioration and movement as well as a statement concerning the watertightness of the structure. Guidelines mandate that the written report be sufficiently detailed so that a comparison with prior reports will show the rate of any deterioration in the condition of the exterior.

The facade inspection must be performed "close-up" to ensure a complete inspection. Such an inspection is defined as one requiring the architect or engineer to review the exterior "from a platform or device while traveling 100 percent of the surface of the exterior walls and enclosures." The inspection must also include a review of the known history of the building and the nature of the materials used in the facade. Special efforts to detect splitting and fracturing of terra cotta and determine the condition of metal anchors and supports are required, and photographs must be taken to document any significant deterioration. A detailed description of the examination undertaken by the design professional must also be included.

Under the Chicago statute, the report of the architect or engineer goes to the owner and the Commissioner of Buildings. The Commissioner will stamp the report and return it to the owner if it is approved. If the report is found to be unsatisfactory, the owner will be required to have the architect or engineer make corrections and resubmit it. Any unsafe conditions discovered in the building will result in notification by the Buildings Department to the owner requiring that immediate repairs be undertaken to ensure code compliance.

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