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BRIDGE INSPECTION, MAINTENANCE, AND REPAIR

DEPARTMENTS OF THE ARMY AND THE AIR FORCE
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CHAPTER 5

BRIDGE CONSTRUCTION MATERIALS

Section I. CONCRETE

5-1. General

Concrete is essentially a compressive material. While it has adequate strength for most structural uses, it is best suited for relatively massive members that transmit compressive loads directly to the founding material. Although concrete has low tensile strength, reinforcing it with steel bars produces a material that is suitable for the construction of flexural members, such as deck slabs, bridge girders, etc. Prestressed concrete is produced by a technique which applies compression to concrete by means of highly stressed strands and bars of high strength steel wire. This compressive stress is sufficient to offset the tensile stress caused by the applied loads. Prestressing has greatly increased the maximum span length of concrete bridges.

5-2. Physical and mechanical properties

a. Strength. Compressive strength is high, but shear and tensile strengths are much lower, being about 12 percent and 10 percent, respectively, of the compressive strength.

b. Porosity. Concrete is inherently porous and permeable since the cement paste never completely fills the spaces between the aggregate particles. This permits absorption of water by capillary action and the passage of water under pressure.

c. Extensibility. Concrete is considered extensible, i.e., undergoes large extensions without cracking. However, this presupposes a high-quality concrete and freedom from restraint.

d. Fire resistance. High-quality concrete is highly resistant to the effects of fire. However, intense heat will damage concrete.

e. Elasticity. Concrete under ordinary loads is elastic, i.e., stress is proportional to strain. Under sustained loads, the elasticity of concrete is significantly lowered due to creep. This makes concrete less likely to crack.

f. Durability. The durability of concrete is affected by climate and exposure. In general, as the water-cement ratio is increased, the durability will decrease correspondingly. Properly proportioned, mixed, and placed, concrete is very durable.

g. Anisotropy. Concrete itself is generally isotropic, but once reinforced with steel bars or prestressed with steel wires, it becomes anisotropic, i.e., its strength varies depending on the direction in which it is loaded.

5-3. Indication and classification of deterioration

While performing an inspection of concrete structures, it is important that the conditions observed be described in very clear and concise terms that can later be understood by others. The common terms used to describe concrete deterioration are discussed:

a. Cracking. Cracks in concrete may be described in a variety of ways. Some of the more common ways are in terms of surface appearance, depth of cracking, width of cracking, current state of activity, and structural nature of the crack:

(1) *Surface appearance.* The surface appearance of cracks can give the first indication of the cause of cracking. Two categories exist:

(a) Pattern or map cracks. These are rather short cracks, usually uniformly distributed and interconnected, that run in all directions (figure 5-1). These cracks are the result of restraint of contraction of the surface layer or possibly an increase of volume in the interior of the concrete. Another type of pattern crack is "D-cracking." These cracks usually start in the lower part of a concrete slab adjacent to joints, where moisture accumulates and progresses away from the corners of the slab (figure 5-2). Vertical cracks near vertical expansion joints in abutments and walls can also be classified as D-cracks.

(b) Individual cracks. These cracks run in definite directions and may be multiple cracks in parallel at definite intervals. Individual cracks indicate tension in the direction perpendicular to the cracking. Several terms may be used to describe the direction that an individual or isolated crack runs: diagonal, longitudinal, transverse, vertical, and horizontal. The directions of these cracks are demonstrated in figure 5-3.

(2) *Depth of cracking.* This category is self-explanatory. The four categories generally used to describe crack depth are surface, shallow, deep, and through.

(3) *Width of cracking.* Three width ranges are used: fine (generally less than 1/32 inch); medium

Figure 5-1. Pattern or map cracking on a pier.

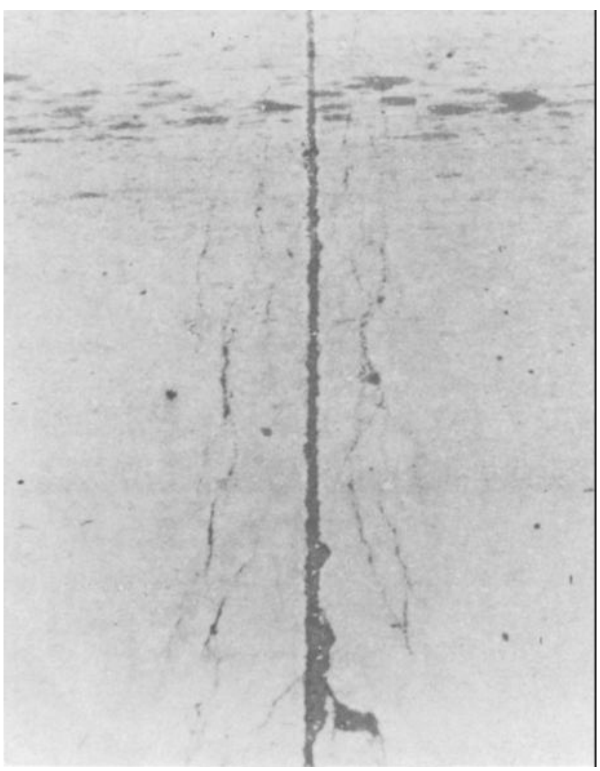

Figure 5-2. D-cracking on a deck.

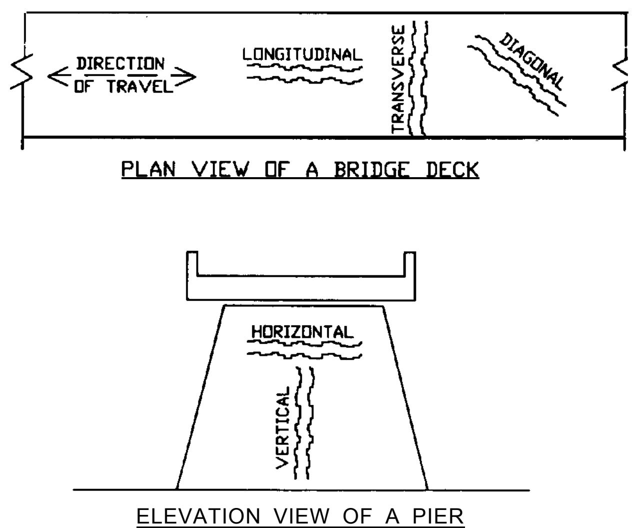

Figure 5-3. Nomenclature for individual cracks.

(between 1/32 and 1/16 inch); and wide (over 1/16 inch).

(4) *Current state of activity.* The activity of the crack refers to the presence of the factor causing the cracking. The activity must be taken into account when selecting a repair method. Two categories exist:

(a) *Active cracks.* These are cracks for which the mechanism causing the cracking is still at work. If the crack is currently moving, regardless of why the crack formed initially or whether the forces that caused it to form are or are not still at work, it must be considered active. Also, any crack for which an exact cause cannot be determined should be considered active.

(b) *Dormant cracks.* These are cracks which are not currently moving or for which the movement is of such magnitude that a repair material will not be affected by the movement.

(5) *Structural nature of the crack.* Cracks may also be categorized as structural (caused by excessive live or dead loads) and nonstructural (caused by other means). Structural cracks will usually be substantial in width, and the opening may tend to increase as a result of continuous loading and creep of the concrete. In general, it can be difficult to determine readily during a visual examination whether a crack is structural or nonstructural. Such a determination will frequently require a structural engineer.

(6) *Combination of descriptions.* To describe cracking accurately, it will usually be necessary to use several terms from the various categories listed. For example: (1) shallow, fine, dormant, pattern cracking, or (2) shallow, wide, dormant, isolated short cracks.

b. *Disintegration.* Disintegration of concrete may be defined as the deterioration of the concrete into small fragments or particles due to any cause. It differs from spalling in that larger pieces of intact concrete are lost when spalling occurs. Disintegration may be caused by a variety of causes including aggressive water attack, freezing and thawing, chemical attack, and poor construction practices. Two of the most commonly used terms used to describe disintegration are scaling and dusting:

(1) *Scaling.* This is the gradual and continuing loss of surface mortar and aggregate over an area. The inspector should describe the character of the scaling, the approximate area involved, and the location of the scaling on the bridge. Scaling should be classified as follows:

(a) *Light scale.* Loss of surface mortar up to ¼ inch deep, with surface exposure of coarse aggregates (figure 5-4), is considered light scale.

(b) *Medium scale.* Loss of surface mortar from ¼ to ½ inch deep, with some added mortar loss between the coarse aggregates (figure 5-5), is considered medium scale.

(c) *Heavy scale.* Loss of surface mortar surrounding aggregate particles of ½ to 1 inch deep is considered heavy scale. Aggregates are clearly exposed and stand out from the concrete (figure 5-6).

(d) *Severe scale.* Loss of coarse aggregate particles as well as surface mortar and the mortar surrounding the aggregates is considered severe scale. Depth of the loss exceeds 1 inch.

Figure 5-4. Light scale.

Figure 5-5. Medium scale.

Figure 5-6. Heavy scale.

(2) *Dusting.* Dusting is the development of a powdered material at the surface of hardened concrete. Dusting will usually be noted on horizontal concrete surfaces that receive a great deal of traffic. Typically, dusting is a result of poor construction practice. For example, sprinkling water on a concrete surface during finishing will frequently result in dusting.

c. Spalling. Spalling is defined as the development of fragments, usually in the shape of flakes, detached from a larger mass. As previously noted, spalling differs from disintegration in that the material being lost from the mass is concrete and not individual aggregate particles that are lost as the binding matrix disintegrates. The distinction between these two symptoms is important when attempting to relate symptoms to causes of concrete problems. Spalls can be categorized as follows:

(1) *Small spall.* These are not greater than ¾ inch in depth nor greater than 6 inches in any dimension (figure 5-7).

(2) *Large spall.* These are deeper than ¾ inch and greater than 6 inches in any dimension (figure 5-8).

(3) *Special case of spalling.* Two special cases of spalling must be noted:

(a) Popouts. These appear as shallow, typically conical depressions in a concrete surface (figure 5-9). They may be the result of freezing of concrete that contains some unsatisfactory aggregate particles. They are easily recognizable by the shape of the pit remaining in the surface and by a portion of the offending aggregate particle usually being visible in the hole.

Figure 5-7. Small spall.

(b) Spalling caused by the corrosion of reinforcement. One of the most frequent causes of spalling is the corrosion of reinforcing steel. During a visual examination, spalling caused by corrosion of reinforcement is usually an easy symptom to recognize since the corroded metal will be visible along with rust staining, and the diagnosis will be straightforward (figure 5-10).

(c) Joint spall. This is an elongated depression along an expansion, contraction, or construction joint (figure 5-11).

5-4. Causes of deterioration

Once the inspection of a concrete structure has been completed, the cause or causes for any deterioration must be established. Since many of the symptoms may be caused by more than one mechanism acting upon the concrete, it is necessary to have an understanding of the basic underlying causes of damage and deterioration. Table 5-1 summarizes the various causes of deterioration in concrete and their associated indicators. These causes are discussed:

a. Accidental loadings. These loadings are generally short-duration, one-time events such as vehicular impact or an earthquake. These loading:

Figure 5-8. Large spall.

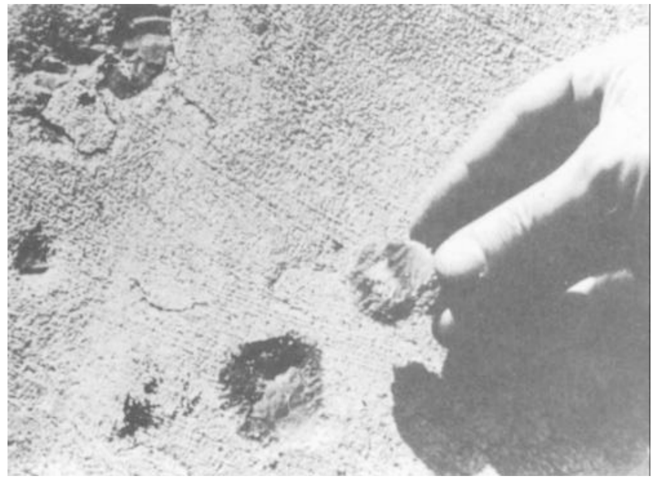

Figure 5-9. Popouts.

can generate stresses higher than the strength of the concrete resulting in localized or general failure. This type of damage is indicated by spalling or cracking of the concrete. Laboratory analysis is generally not necessary.

b. Chemical *reactions.* This category includes several specific causes of deterioration that exhibit a wide variety of symptoms as described:

(1) *Acid attack.* Portland cement is generally not very resistant to attack by acids, although weak acids can be tolerated. The products of combustion of many fuels contain sulfurous gases which combine with moisture to form sulfuric acid.

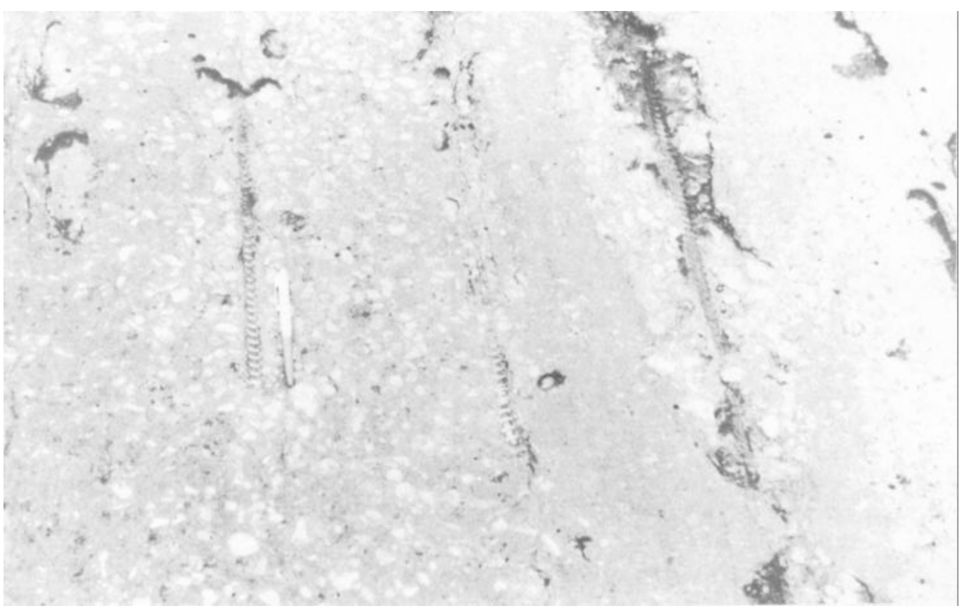

Figure 5-10. Spa11 due to reinforcement corrosion.

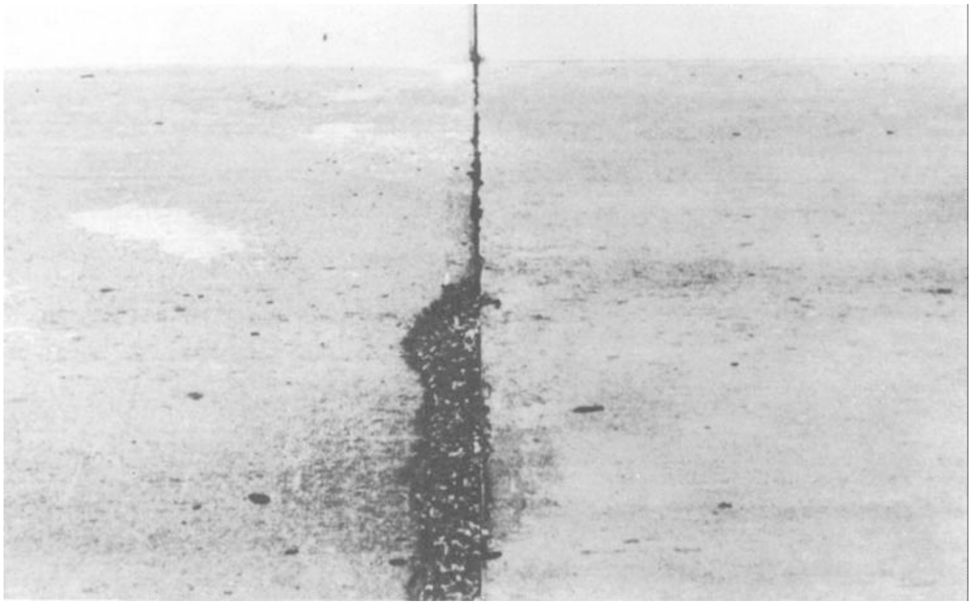

Figure 5-11. Joint spall.

Table 5-1. Relation of symptoms to causes of distress and deterioration of concrete

Symptoms	Construction Faults	Cracking	Disintegration	Distortion/ Movement	Erosion	Joint Failures	Seepage	Spalling
Accidental loadings		X						X
Chemical reactions		X	X				X	
Construction errors	X	X				X	X	X
Corrosion		X						X
Design errors		X				X	X	X
Erosion			X		X			
Freezing and thawing		X	X					X
Settlement and movement		X		X		X		
Shrinkage		X						
Temperature changes		X				X		X

Other possible sources for acid formation are sewage, some peat soils, and some mountain water streams. Visual examination will show disintegration of the concrete leading to the loss of cement paste and aggregate from the matrix. If reinforcing steel is reached by the acid, rust staining, cracking, and spalling may be seen. If the nature of the solution in which the deteriorated concrete is located is unknown, laboratory analysis can be used to identify the specific acid involved.

(2) *Alkali-carbonate rock reaction.* Certain aggregates of carbonate rock have been reactive in

concrete. The results of these reactions have been both beneficial and destructive. Visual examination of those reactions that are serious enough to disrupt the concrete in a structure will generally show map or pattern cracking and a general appearance which indicates swelling of the concrete. This reaction is distinguished from that of the alkali-silica reaction by the lack of silica gel exudations at cracks. Petrographic examination may be used to confirm the presence of alkali-carbonate rock reaction.

(3) *Alkali-silica reaction.* Some aggregates containing silica that is soluble in highly alkaline solutions may react to form expansive products which will disrupt the concrete. This reaction is indicated by map or pattern cracking and a general appearance of swelling of the concrete. Petrographic examination may be used to confirm the presence of this reaction.

(4) *Miscellaneous chemical attack.* Concrete will resist chemical attack to varying degrees depending upon the exact nature of the chemical. Concrete which has been subjected to chemical attack will usually show surface disintegration and spalling and the opening of joints and cracks. There may also be swelling and general disruption of the concrete mass. Aggregate particles may be seen protruding from the remaining concrete mass.

(5) *Sulfate attack.* Naturally occurring sulfates of sodium, potassium, calcium, or magnesium are sometimes found in soil or in solution in ground water adjacent to concrete structures. The reactions involving these sulfates result in an increase in volume of the concrete. Visual inspection will show map and pattern cracking as well as a general disintegration of the concrete. Laboratory analysis can verify the occurrence of the reactions described.

c. Construction errors. Failure to follow specified procedures and good practice or outright carelessness may lead to a number of conditions that may be grouped together as construction errors. Typically, most of these errors do not lead directly to failure or deterioration of concrete. Instead, they enhance the adverse impacts of other mechanisms identified in this chapter. The following are some of the most common construction errors:

(1) *Addition of water to concrete.* The addition of water while in the delivery truck will often lead to concrete with reduced strength and durability. The addition of water while finishing a slab will cause crazing and dusting of the concrete in service.

(2) *Improper consolidation.* Improper consolidation of concrete may result in a variety of defects, the most common being bugholes, honeycombing, and cold joints. These defects make it much easier for any damage-causing mechanism to initiate deterioration of the concrete.

(3) *Improper curing.* Unless concrete is given adequate time to cure at a proper humidity and temperature, it will not develop the characteristics that are expected and that are necessary to provide durability. Symptoms of improperly cured concrete can include various types of cracking and surface disintegration. In extreme cases where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

(4) *Improper location of reinforcing steel.* This section refers to reinforcing steel that is either improperly located or is not adequately secured in the proper location. Either of these faults may lead to two general types of problems. First, the steel may not function structurally as intended resulting in structural cracking or failure. The second type of problem stemming from improperly located or tied steel is one of durability. The tendency seems to be for the steel to end up close to the surface of the concrete. As the concrete cover over the steel is reduced, it is much easier for corrosion to begin.

d. Corrosion of embedded metals. Under most circumstances, Portland-cement concrete provides good protection to the embedded reinforcing steel. This protection is generally attributed to the high alkalinity of the concrete adjacent to the steel and to the relatively high electrical resistivity of the concrete. However, this corrosion resistance will generally be reduced over a long period of time by carbonation, and the steel will begin to corrode. Deicing salts are the most common cause of the corrosion. Corrosion of the steel will cause two things to occur. First, the cross-sectional capacity of the reinforcement is reduced which in turn reduces the load-carrying capacity of the steel. Second, the products of the corrosion expand since they occupy about eight times the volume of the original material. This leads to cracking and ultimately spalling of the concrete. For mild steel reinforcing, the damage to the concrete will become evident long before the capacity of the steel is reduced enough to affect its load-carrying capacity. However, for prestressing steel slight reductions in section can lead to catastrophic failure. Visual examination will typically reveal rust staining of the concrete. This staining will be followed by cracking. Cracks produced by corrosion usually run in straight, parallel lines at uniform intervals corresponding to the spacing of the reinforcement. As deterioration continues, spalling of the concrete over the reinforcing steel will occur with the reinforcing bars becoming visible. A laboratory analysis may be beneficial to determine the chloride contents in the concrete throughout its depth. This procedure may be used to determine the amount of concrete to be removed during a rehabilitation project.

e. Design errors. Design errors generally result from inadequate structural design or from lack of attention to relatively minor design details:

(1) *Inadequate structural design.* This will cause cracking and/or spalling in areas which are subject to the highest stresses. To identify this as a source of damage, the locations of the damage should be compared to the types of stresses that should be present in the concrete. A detailed structural analysis may be required, and thus a qualified structural engineer should be consulted if this problem is apparent.

(2) *Poor design details.* Poor detailing may result in localized concentrations of high stresses in otherwise satisfactory concrete. The following are some of the more design detail problems:

(a) Abrupt changes in section. This may cause stress concentrations that may result in cracking. Typical examples would include the use of relatively thin bridge decks rigidly tied into massive abutments or patches and replacement concrete that are not uniform in plan dimensions.

(b) Reentrant corners and openings. These locations are subject to stress concentrations, and when insufficiently reinforced, cracking may occur.

(c) Inadequate drainage. This will cause ponding of water, which may result in excessive loading or, more likely, leakage or saturation of concrete. Concrete subject to freeze-thaw cycles is especially vulnerable to this type of damage.

(d) Insufficient travel in expansion joints. Inadequately designed expansion joints may result in spalling of concrete adjacent to the joints.

(e) Rigid joints between precast units. Designs utilizing precast elements must provide for movement between adjacent precast elements or between the precast elements and the supporting frame. Failure to provide for this movement can result in cracking or spalling.

f. Wear and abrasion. Traffic abrasion and impact cause wearing of bridge decks; while curbs, parapets, and piers are damaged by the scraping action of such vehicles as snow plows and sweepers. Deck wear also appears as cracking and ravelling at joint edges.

g. Freezing and thawing. The cyclic freezing and thawing of critically saturated concrete will cause its deterioration. Deicing chemicals may also accelerate the damage and lead to pitting and scaling. This damage ranges from surface scaling to extensive disintegration. Laboratory examination of concrete cores with this damage will often show a series of cracks parallel to the surface of the structure.

h. Foundation movement. These movements will cause serious cracking in structures. Further discussion of this problem is provided in section VII of this chapter.

i. Shrinkage. Shrinkage is caused by the loss of moisture from concrete. It may be divided into two categories: that which occurs before setting (plastic shrinkage) and that which occurs after setting (drying shrinkage). Cracking due to plastic shrinkage will be seen within a few hours of concrete placement. The cracks are generally wide and shallow and isolated rather than patterned. Cracks due to drying shrinkage are characterized by their fineness and absence of any indication of movement. They are usually shallow, a few inches in depth, and in an orthogonal or blocky pattern.

j. Temperature changes. Changes in temperature cause a corresponding change in volume of the concrete, and when sufficiently restrained against expansion or contraction cracking will occur. Temperature changes will generally result from the heat of hydration of cement in large concrete placements, variations in climatic conditions, or fire damage.

5-5. Assessment of concrete

Assessment of existing reinforced concrete in bridges is basically associated with fully identifying the cause and extent of observed or suspected deterioration. The method of description of damaged concrete and the possible causes for deterioration were discussed in paragraph 5-5. This section will provide guidance concerning the available test methods for determining the causes of the deterioration and quantifying its extent. The range of available test methods is large and includes in situ nondestructive tests upon the actual structure as well as physical, chemical, and petrographic tests upon samples removed from the structure, and load testing. Table 5-2 summarizes basic characteristics of the most widely established test methods classified according to the features which may be assessed most reliably in each case. An overview of some of the more common tests follows.

Table 5-2. Test methods for concrete

Property Under Investigation	Test	ASTM Designation	Equipment Type
Corrosion of embedded steel	Half-cell potential	C876	Electrical
	Resistivity		Electrical
	Cover depth		Electromagnetic
	Carbonation depth		Chemical and microscopic
	Chloride penetration		Chemical and microscopic
Concrete quality, durability and deterioration	Rebound hammer	C805	Mechanical
	Ultrasonic pulse velocity	C597	Electronic
	Radiography		Radioactive
	Radiometry		Radioactive
	Permeability		Hydraulic
	Absorption		Hydraulic
	Petrographic	C856	Microscopic
	Sulphate content		Chemical
	Expansion		Mechanical
	Air content	C457	Microscopic
	Cement type and content	C85, C1084	Chemical and microscopic
Concrete strength	Cores	C42, C823	Mechanical
	Pullout	C900	Mechanical
	Pulloff		Mechanical
	Breakoff		Mechanical
	Internal fracture		Mechanical
	Penetration resistance	C803	Mechanical
Integrity and structural performance	Tapping		Mechanical
	Pulse-echo		Mechanical/electronic
	Dynamic response	C215	Mechanical/electronic
	Thermography		Infrared
	Strain or crack measurement		Optical/mech./elec.

Additional references should be consulted prior to actual usage of these tests.

a. Core drilling. Core drilling to recover concrete for laboratory analysis or testing is the best method of obtaining information on the condition of concrete within a structure. However, since core drilling is expensive and destructive, it should be considered only when sampling and testing of interior concrete is deemed necessary. The core samples should be sufficient in number and size to permit appropriate laboratory examination and testing. For compressive strength, static or dynamic modulus of elasticity, the diameter of the core should not be less than three times the nominal maximum size of aggregate. Warning should be given against taking NX size (2 1/8-inch diameter) cores in concrete containing 2- to 6-inch maximum size aggregate. Due to the large aggregate size, these cores will generally be recovered in short broken pieces. When drilling in poor-quality concrete with any size core barrel, the material will generally come out as rubble. When drill hole coring is not practical or core recovery is poor, a viewing system such as a borehole camera, borehole television, or borehole televiewer may be used for evaluating the interior concrete conditions. In addition, some chemical tests may be performed on smaller drilled powdered samples from the structure, thus causing substantially less damage than that produced by coring, but the likelihood of sample contamination is increased and precision may be reduced.

b. Laboratory investigations. Once samples of concrete have been obtained, whether by coring, drilling, or other means, they should be examined in a qualified laboratory. In general, the examination will include one or more of the following examinations:

(1) *Petrographic examination.* This type of examination may include visual and microscopic inspection, x-ray diffraction analysis, differential thermal analysis, x-ray emission techniques, and thin section analysis. These techniques may be expected to provide information on the following: aggregate condition; pronounced cement-aggregate reactions; deterioration of aggregate particles in place; denseness of cement paste; homogeneity of the concrete; depth and extent of carbonation; occurrence and distribution of fractures; characteristics and distribution of voids; and presence of contaminating substances.

(2) *Chemical analysis.* Chemical analyses of hardened concrete or of selected portions (paste, mortar, aggregate, reaction products, etc.) may be used to estimate the cement content, original water-cement ratio, and the presence and amount of chloride and other admixtures. The chloride analysis is the most common of these analyses. It is used to provide a quantitative measure of chloride ion contamination and, thus, the potential for active steel corrosion at various levels in the concrete deck. Samples for this test are usually taken by a rotary hammer drill. The "threshold" chloride content, or amount of chloride needed to initiate corrosion, is approximately 2.0 pounds of chloride content per cubic yard of concrete.

(3) *Physical analysis.* The following physical and mechanical tests are generally performed on concrete cores: density, compressive strength, modulus of elasticity, Poisson's ratio, pulse velocity, and volume change potential by freezing and thawing.

c. Nondestructive testing (NDT). The purpose of NDT is to determine the various properties of concrete such as strength, modulus of elasticity, homogeneity, and integrity, as well as conditions of strain and stress, without damaging the structure. Some of the most commonly used tests are discussed:

(1) *Rebound number (hammer).* Rebound numbers may be used to estimate the uniformity and quality of in situ concrete. The rebound number is obtained by the use of a special "hammer" that consists of a steel mass and a tension spring in a tubular frame. The measured rebound number can be related to calibration curves which will give an indication of the in situ concrete strength. The rebound number increases with the strength of the concrete. This method is inexpensive and allows for a large number of measurements to be rapidly taken so that large exposed areas of concrete can be mapped within a few hours. It is, however, a rather imprecise test and does not provide a reliable prediction of the strength of concrete. The measurements can be affected by: smoothness of the concrete surface; moisture content of the concrete; type of coarse aggregate; size, shape, and rigidity of the specimen; and carbonation of the concrete surface.

(2) *Penetration resistance (probe).* This test is also used for a quick assessment of quality and uniformity of concrete. The apparatus most often used for penetration resistance is the Windsor Probe, a special gun which uses a 32-caliber blank with a precise quantity of powder to fire a high-strength steel probe into the concrete. The depth of penetration of the probe into the concrete can then be related by calibration curves to concrete compressive strength. A probe will penetrate deeper as the density, subsurface hardness, and strength of the concrete decrease. It should not be considered for use as a precise measurement of concrete strength. However, useful estimates of the compressive strength may be obtained if the probe is properly calibrated. This test does damage the concrete, leaving a hole of about 0.32 inches in diameter for the depth of the probe, and may cause minor cracking and some surface spalling. Minor repairs of exposed surfaces may be necessary.

(3) *Ultrasonic pulse velocity.* This method involves the measurement of the time of travel of electronically pulsed compressional waves through a known distance in concrete. These velocities can be used to assess the general condition and quality of concrete, to assess the extent and severity of cracks in concrete, and to delineate areas of deteriorated or poor-quality concrete. Good-quality, continuous concrete will normally produce high velocities accompanied by good signal strengths. Poor-quality or deteriorated concrete will usually decrease velocity and signal strength. Concrete of otherwise good quality, but containing cracks, may produce high or low velocities, depending upon the nature and number of cracks, but will almost always diminish signal strength. This method does not provide a precise estimate of concrete strength. Moisture variations and the presence of reinforcing steel can affect the results. Skilled personnel are required in the analysis of the results.

(4) *Surface tapping (chain drag).* Experience has shown that the human ear, used in conjunction with surface tapping, is the most efficient and economical method of determining major delamination in bridge decks. Chain dragging is the most commonly used method for this purpose. This is, however, a very subjective test in that the operator must be able to differentiate between sound and unsound regions, and the results cannot be easily quantified.

d. Steel corrosion assessment. The most commonly used test for assessing the current state of reinforcing steel corrosion is the half-cell potential test. This test involves measurement of the electrical potential of an embedded reinforcing bar relative to a reference half-cell placed on the concrete surface. Potential differences more negative than - 0.35 volts indicates a high degree of probability of active corrosion of the reinforcing steel. Potential readings of - 0.20 volts and lower indicate the probability of inactive or no corrosion, while readings between -0.20 and -0.35 volts indicate the possibility of active corrosion.

e. *Load tests.* Occasionally it may be necessary to examine the overall behavior of an entire bridge structure or section of a bridge. This may be achieved electronically by measuring the response to dynamic loading with the aid of appropriately positioned accelerometers or alternatively monitoring the performance under static test loads. The most common method however, is to measure strains and deflections (with precise leveling or lasers) produced from static full-scale test loads. These tests are generally expensive but yield valuable information as to the overall "health" of a structure. This type of test can be conducted on any type of bridge, regardless of the material type.

Section II. STRUCTURAL STEEL

5-6. Physical and mechanical properties

a. Strength. Steel possesses tremendous compressive and tensile strength and is highly resistant to shear forces. Thin steel sections, however, are vulnerable to compressive buckling.

b. Ductility. Both the low-carbon and low-alloy steels normally used in bridge construction are quite ductile. Brittleness may occur because of heat treatment, welding, or through metal fatigue.

c. Durability. Steel, when protected properly, is extremely durable.

d. Fire resistance. Steel is subject to a loss of strength when exposed to high temperatures such as those resulting from fire.

e. Corrosion. Unprotected carbon steel corrodes (rusts) readily. However, it can readily be protected.

f. Weldability. Although steel is weldable, it is necessary to determine the chemistry of the steel and to select a suitable welding procedure before starting welding operations on a bridge.

g. Others. Steel is elastic and conducts heat and electricity.

5-7. Indicators and classification of deterioration

a. Rust. Rusted steel varies in color from dark red to dark brown. Initially, rust is fine grained, but as it progresses it becomes flaky or scaly in character. Eventually, rust causes a pitting of the member. The inspector should note the location, characteristics, and the extent of the rusted areas. The depth of heavy pitting should be measured and the size of any perforation caused by rusting should be recorded. Rust may be classified as follows:

(1) *Light.* A light, loose rust formation pitting the paint surface.

(2) *Moderate.* A looser rust formation scales or flakes forming. Definite areas of rust are discernible.

(3) *Severe.* A heavy, stratified rust or rust scale with pitting of the metal surface. This rust condition eventually culminates in the perforation of the steel section itself.

b. Cracks.

(1) Cracks in the steel may vary from hairline thickness to sufficient width to transmit light through the member. The first visible evidence is normally a crack in the paint film. Depending on the location and the length of time the paint has been open, there may be a thin line of rust stain emanating from the crack as shown in figure 5-12. Crack identification on unpainted A588 steel is particularly difficult. There is no staining due to oxidation and the rough surface texture tends to hide the crack.

(2) Any type of crack is obviously serious and should be reported at once. Record the location and length of all cracks and indicate whether the cracks are open or closed. The full length of the crack may not be completely visible. A suitable nondestructive test such as the dye penetrant test (discussed in the following section) can help to establish its full length.

(3) Cracks in fracture critical steel members (which should have been identified prior to the inspection) are especially serious and therefore these members should be inspected with extreme care. Cracks of any size should be immediately reported to the appropriate authority.

Figure 5-12. Rust stained crack in steel girder.

c. Buckles and kinks. These conditions develop mostly because of damage arising from thermal strain, overload, or added load conditions. The latter condition is caused by failure or the yielding of adjacent members or components. Collision damage may also cause buckles, kinks, and cuts (figure 5-13). Look for cracks radiating from cuts or notches. Note the members damaged, the type, location, and extent of the damage, and measure the amount of deformation, if possible.

d. Stress concentrations. Observe the paint around the connections at joints for fine cracks as indications of large strains due to stress concentrations. Be alert for sheared or deformed bolts and rivets.

e. Galvanic corrosion. This condition will appear essentially similar to rust.

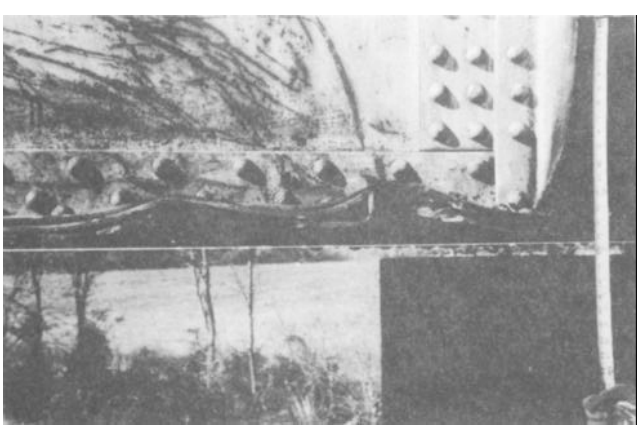

Figure 5-13. Buckled flange due to collision.

5-8. Causes of deterioration

a. Air and moisture. Air and moisture cause rusting of steel, especially in a marine climate.

b. Industrial fumes. Industrial fumes in the atmosphere, particularly hydrogen sulfide, cause deterioration of steel.

c. Deicing agents. While all deicing agents attack steel, salt is the most commonly encountered chemical on bridges.

d. Seawater and mud. Unprotected steel members, such as piles immersed in water and embedded in mud, undergo serious deterioration and loss of section.

e. Thermal strains or overloads. Where movement is restrained, or where members are overstressed, the steel may yield, buckle, or crack (or rivets and bolts may shear).

f. Fatigue and stress concentrations. Cracks may develop because of fatigue or poor details which produce high stress concentrations. Examples of such details are: reentrant corners, abrupt and large changes in plate widths and/or thicknesses, a concentration of heavy welds, or an insufficient bearing area for a support. Fatigue and stress concentrations are very important factors in the failure of steel structures. A thorough discussion of these factors is provided in the American Association of State Highway Transportation Officials (AASHTO) Manual, "Inspection of Fracture Critical Bridge Members."

g. Fire. Extreme heat will cause serious deformations of steel members.

h. Collisions. Trucks, over-height loads, derailed cars, etc., may strike steel beams or columns, damaging the bridge.

i. Animal wastes. These may cause rusting and can be considered as a special type of direct chemical attack.

j. Welds. Where the flux is not neutralized, some rusting may occur. Welds may crack because of poor welding techniques or poor weldability of the steel. Problems with welds will also be discussed in more detail in chapter 6.

k. Galvanic action. Other metals that are in contact with steel may cause corrosion similar to rust.

5-9. Assessment of deterioration

a. As discussed in paragraph 5-9, the deterioration of steel members is mainly due to rusting or cracking. The determination of section loss due to rusting is of primary importance for a proper load rating assessment to be performed. This is usually done with precise mechanical (such as calipers) or electrical (such as a "depth meter") measuring devices. The full assessment of cracks in steel members is very important since they can cause rapid failure to some members. All of the cracks in a member should be located and their extent of propagation should be fully defined. In addition to a close visual examination, a wide variety of nondestructive test methods exists for these purposes and several of the most common methods are briefly discussed:

(1) *Dye penetrant.* This test is used to identify the location and extent of surface cracks and surface defects, such as hairline fatigue cracks. It cannot be used to locate subsurface defects. For the test, the area must be thoroughly cleaned of paint, rust, scale, grease, and oily films. Then, an oil-based liquid penetrant is applied which is intended to be absorbed into any cracks present. After a specific amount of time, the excess is wiped off and a developer applied. The developer acts as a blotter, drawing out a portion of the penetrant which has seeped into the defect, causing a bright red outline of the defect to appear in the developer.

(2) *Magnetic particle testing.* In this test, a magnetic field is induced in the steel by means of a moderately sized power source. Detection of a flaw is accomplished by application of inert compounds of iron which are attracted to the magnetic field as it leaves and then reenters the steel in the area of the flaw. This test requires a highly trained inspector.

(3) *Radiographic inspection.* This process involves the application of x rays to an area or specimen in question. The ability of the specimen to dilute the density of the x rays passing through indicates its relative homogeneity. Any discontinuity, such as a fatigue crack, will show up on film placed behind the specimen as less dense than the sound material. This test method is most beneficial and has been used most successfully in analyzing welds for incomplete fusion, slag and other inclusions, incomplete penetration, and gas pockets.

b. Tensile coupons. To perform an accurate analysis of the bridge's load capacity, the material properties of the steel must be known. In many older bridges, the type of steel, and thus its properties, may not be known. In these cases, the cutting of "coupons" for testing may be necessary. Since this operation causes considerable damage to the member from which the coupon is taken, extreme care must be exercised. The location for the coupon should be carefully chosen to provide the most accurate information while causing the least structural damage to the structure. Guidance from a structural engineer should be obtained. The samples should be 9 to 12 inches long and 2 to 3 inches wide. The actual coupons will be machined from these samples.

Section III. TIMBER

5-10. Physical and mechanical properties

a. Strength. Timber, while not as strong as steel, approximates ordinary concrete in compressive strength. Rated strongest in flexural strength, timber has an allowable compressive strength (parallel to grain) of about 75 percent of the flexural value. Perpendicular to the grain, compressive strength is only 20 percent of the flexural strength. Horizontal shear is limited to 10 percent of the flexural strength.

b. Porosity. Being a cellular, organic material, timber is quite porous.

c. Anisotropy. As may be deduced from the differences in allowable compressive strengths, wood is anisotropic, i.e., it has different strength properties depending upon the manner and direction of loading.

d. Impact resistance. Since timber is able to withstand a greatly increased load momentarily, neither impact nor fatigue are serious problems with timber.

e. Durability. Under certain conditions, and when properly treated or protected, timber is quite durable. However, timber is not a particularly durable material under all conditions. It should be noted that some preservative treatments reduce the strength of timber.

f. Fire resistance. Timber is very vulnerable to damage by fire.

g. Other. Timber is also elastic, low in thermal and electrical conductivity, and subject to volume changes.

5-11. Deterioration: indicators and causes

a. Fungus decay. Fungi usually require some moisture to exist. As a rule, fungus decay can be avoided only by excellent preservative treatment. Fungus decay is classified as follows:

(1) *Mild.* Mild fungus decay appears as a stain or discoloration. It is hard to detect and even harder to distinguish between decay fungi and staining fungi.

(2) *Advanced.* Wood darkens further and shows signs of definite disintegration, with the surface becoming punky, soft and spongy, stringy, or crumbly, depending upon the type of decay or fungus (figures 5-14 and 5-15). It is similar to dryrot of door posts and outside porches. Fruiting bodies of fungi, similar to those seen on old stumps, may develop. The inspector should note the location, depth of penetration, and size of the areas of decay. Where decay occurs at a joint or splice, the effect on the strength of the connection should be indicated. A knife, icepick, or an increment borer can be used to test for decayed wood. Decay is very likely to occur at connections, splices, support points, or around bolt holes. This may be due to the tendency of such areas to collect and retain moisture or to bolt holes or cuts being made in the surface after the preservative treatment has been applied. Unless these surfaces are subsequently protected, decay is very likely. Any holes, cuts, scrapes, or other breaks in the timber surface which would break the protective layers of the preservative treatment and allow access to untreated wood should be noted.

b. Vermin. The following vermin tunnel in and hollow out the insides of timber members for food and/or shelter:

(1) *Termites.* All damage is inside the surfaces of the wood; hence, it is not visible. White mud shelter tubes or runways extending up from the earth to the wood and on the side of masonry substructures are the only visible signs of infestation. If the timber members exhibit signs of excessive sagging or crushing, check for termite damage with an ice pick or an increment borer.

(2) *Powder-post beetles.* The outer surface is poked with small holes. Often a powdery dust is dislodged from the holes. The inside may be completely excavated.

(3) *Carpenter ants.* Accumulation of sawdust on the ground at the base of the timber is an indicator. The large, black ants may be seen in the vicinity of the infested wood.

(4) *Marine borers.* The inroads of marine borers will usually be most severe in the area between high and low water since they are waterborne, although damage may extend to the mud

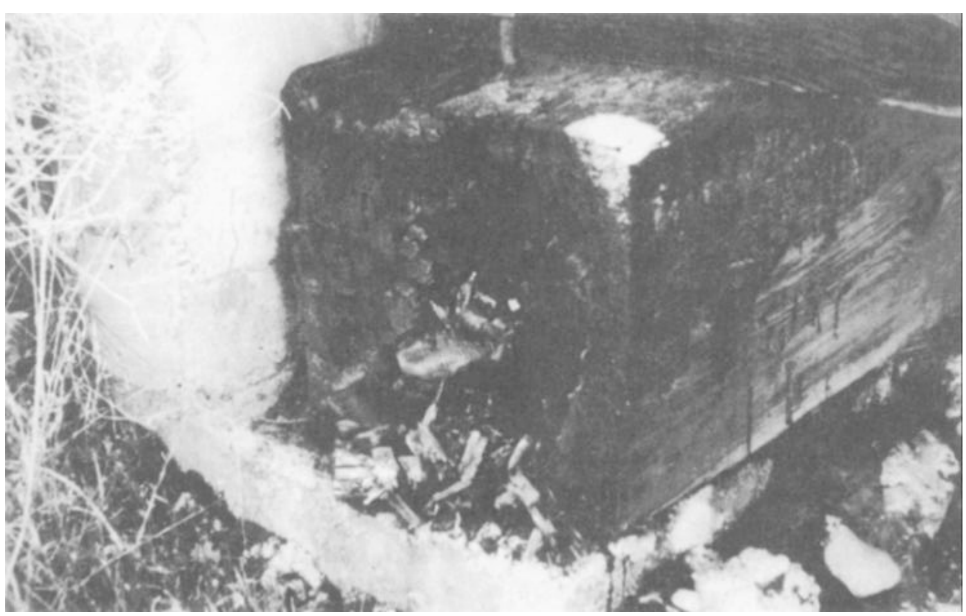

Figure 5-14. Advanced wood decay in crosstie.

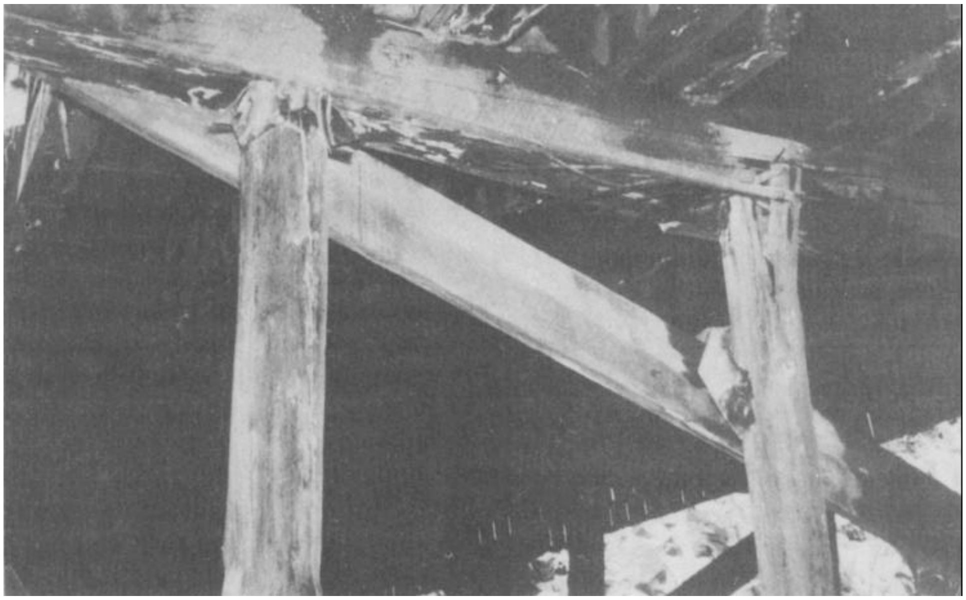

Figure 5-15. Advanced wood decay in bent cap.

line. Where piles are protected by concrete or metal shielding, the shields should be inspected carefully for cracks or holes that would permit entrance of the borers. Unplugged holes such as those left by test borings, nails, bolts, and the like, also permit entrance of these pests. In such cases, there are often no outside evidences of borer attack. The inspector should list the location and extent of damage and indicate whether it is feasible to exterminate the infestation and strengthen the member or if immediate replacement is necessary.

(5) *Mollusk borers (shipworms).* The shipworm is one of the most serious enemies of marine timber installations. The most common species of shipworms is the teredo. This shipworm enters the timber in the early stage of life and remains there for the rest of its life. Teredos reach a length of 15 inches and a diameter of 3/8 inch, although some species of shipworm grow to a length of 6 feet. The teredo maintains a small opening in the surface of the wood to obtain nourishment from the seawater.

(6) *Crustacean borers.* The most commonly encountered crustacean borer is the linoria or wood louse. It bores into the surface of the wood to a shallow depth. Wave action or floating debris breaks down the thin shell of timber outside the borers' burrows, causing the linoria to burrow deeper. The continuous burrowing results in a progressive deterioration of the timber pile cross section which will be most noticeable by the hourglass shape developed between the tide levels.

c. Weathering and warping. This is caused by repeated dimensional changes in the wood, usually due to repeated wetting. It may be described as follows:

(1) *Slight.* Surfaces of wood are rough and corrugated, and the members may even warp (figure 5-16).

(2) *Advanced.* Large cracks extend deeply or completely through the wood (figure 5-16). Wood is crumbly and obviously deteriorated.

d. Chemical attack. Chemicals act in three different ways: a swelling and resultant weakening of the wood, a hydrolysis of the cellulose by acids, or a delignification by alkalis. Animal wastes are also a problem. Chemical attack will resemble decay and should be classified similarly.

e. Fire. Timber is particularly vulnerable to fire. This type of damage is easily recognized and usually will have been reported prior to the inspection.

f. Abrasion and mechanical wear. Wear due to abrasion is readily recognized by the gradual loss of section at the points of wear (figure 5-16). This type of wear is most serious when combined with decay which softens or weakens the wood. The decks of bridges are especially vulnerable. The inspector should report the location, the general area subject to wear, and the loss in thickness. He should also indicate whether immediate remedial action is necessary.

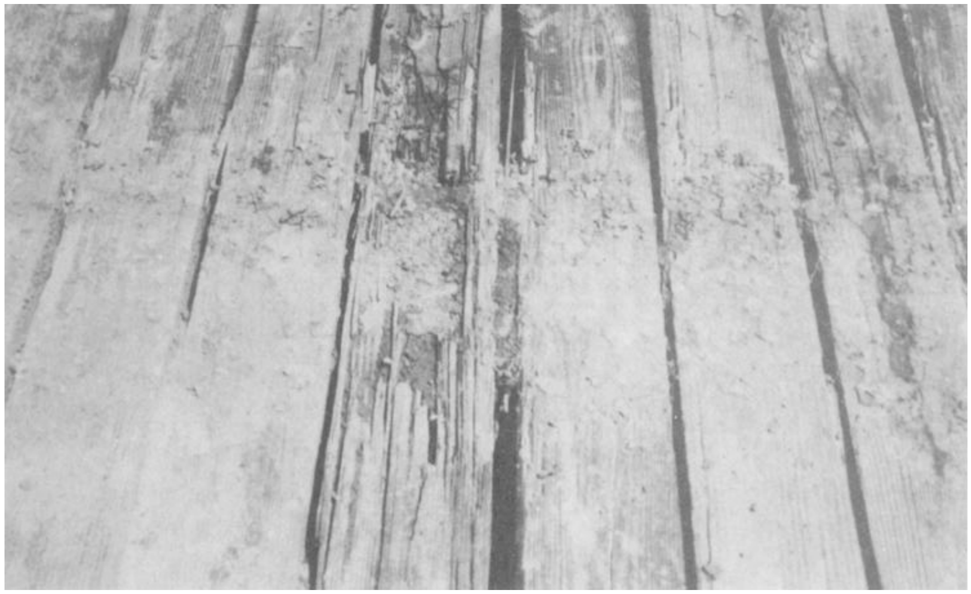

Figure 5-16. Slight and advanced wear on a timber deck (mechanical abrasion wear also present).

g. Collision or overloading damage. Damage will be evident in the form of shattered or injured timbers, sagging or buckled members (figure 5-17), or timbers with large longitudinal cracks (figure 5-18). The inspector should give the location and extent of damage and determine whether immediate remedial action is required.

h. Unplugged holes. Holes left by test borings, nails, bolts, etc. will inevitably allow attack from many of the previously mentioned sources. Their location and extent should be noted and recommendations should be made for their repair.

5-12. Assessment of deterioration

A chipping hammer, an ice pick, and an increment borer are the primary tools used for assessment of wood deterioration. The soundness of all timbers should be first checked by tapping with the hammer and listening for a "hollow" sound. When suspect areas are found, the ice pick should be used to verify the existence of soft spots. If extensive damage is suspected, an increment borer should be used to take a test boring and fully define the extent of deterioration. Borings should be taken very selectively to not further weaken the already damaged timber, and borings should not be made at all if no deterioration is evidenced. Once the boring has been made, save the boring sample and make sure that creosote plugs are inserted into the hole made by the borer.

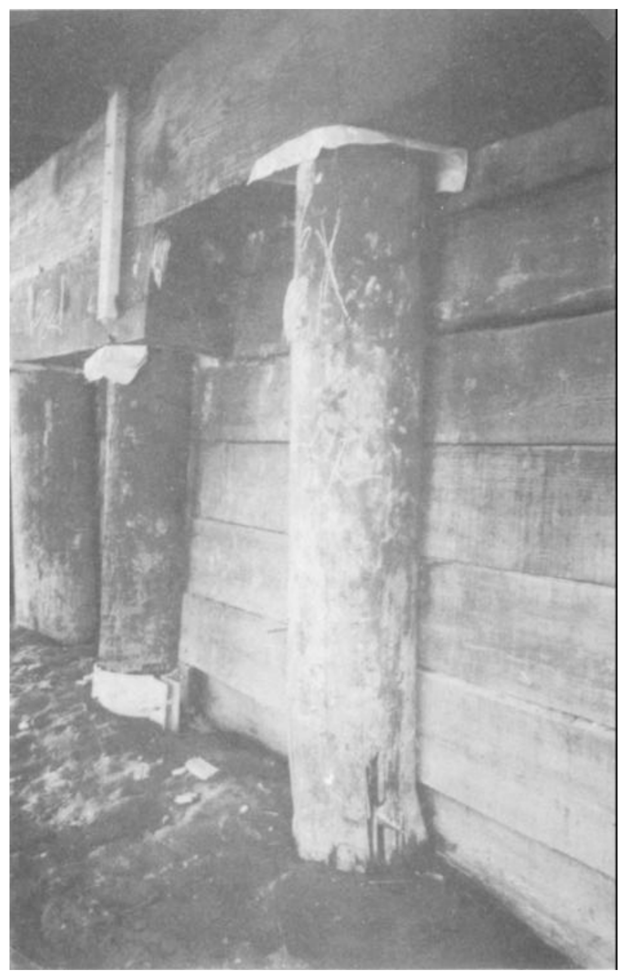

Figure 5-17. Buckled timber pile due to overload.

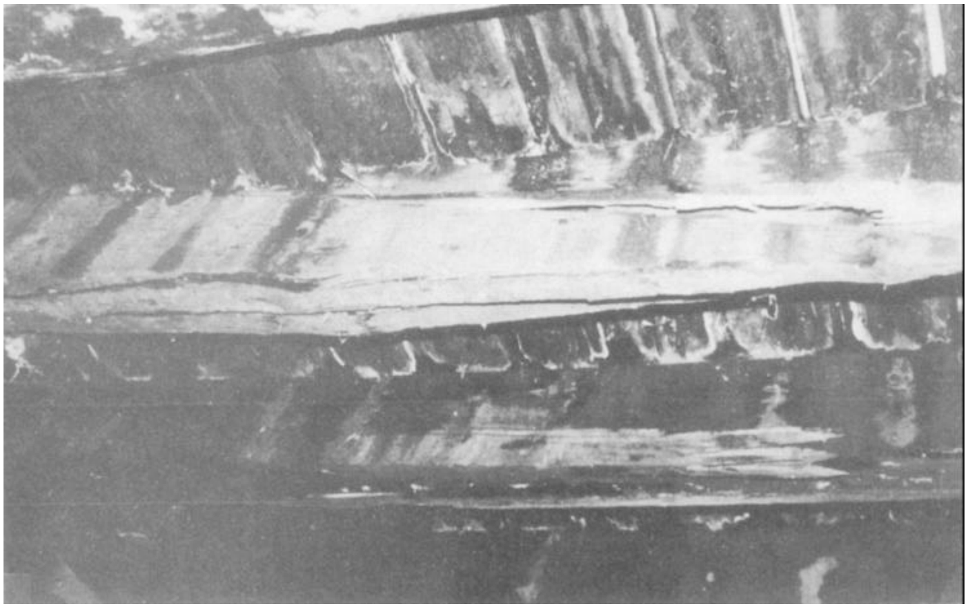

Figure 5-18. Longitudinal cracks in timber beam due to overload.

Section IV. WROUGHT AND CAST IRON

5-13. General

a. Wrought iron is a metal in which slag inclusions are rolled between the microscopic grains of iron. This results in a fibrous material with properties quite similar to steel, although tensile strength is lower than it is in steel.

b. Cast iron is iron in which carbon has been dissolved. Other elements which affect the properties of cast iron are silicon and manganese. In general, a wide range of properties are obtained depending upon the alloying elements used.

5-14. Physical and mechanical properties

a. Wrought iron.

(1) *Strength.* Wrought iron has an ultimate tensile strength of about 50,000 pounds per square inch. However, the rolling process and presence of the slag inclusions make wrought iron anisotropic; wrought iron has a tensile strength across the grain of about 75 percent of its longitudinal strength. While this characteristic has been largely eliminated in the wrought iron made today, wrought iron in the old bridge will be anisotropic.

(2) *Elasticity.* Wrought iron has an elastic modulus of 24,000,000 to 29,000,000 pounds per square inch. This is nearly as high as steel.

(3) *Ductility.* Wrought iron is generally ductile, although its ductility depends, to large extent, upon the method of manufacture.

(4) *Toughness and impact resistance.* Wrought iron is tough and resistant to impact.

(5) *Corrosion resistance.* The fibrous nature of wrought iron produces a tight rust which is less likely to progress to flaking and scaling than is rust on carbon steel.

(6) *Weldability.* Wrought iron is welded with no great difficulty. However, care should be exercised when planning to weld the metal of an existing bridge.

b. Cast iron.

(1) *Strength.* The tensile strength of cast iron varies from 20,000 to 60,000 pounds per square inch, depending upon its composition. In dealing with the iron in old bridges, it must be assumed that the cast iron will be near the lower end of the scale. The compressive strength of cast iron is high, from 60,000 pounds per square inch higher. For this reason, cast iron was used for compression members in the early iron bridges, while wrought iron was used for tension members.

(2) *Elasticity.* Cast iron has an elastic modulus of 13,000,000 pounds per square inch.

(3) *Brittleness.* Cast iron is brittle.

(4) *Impact resistance.* Cast iron possesses good impact resistant properties.

(5) *Corrosion resistance.* Cast iron, in general, is more corrosion resistant than the other ferrous metals.

(6) *Weldability.* Due to its high carbon content, cast iron is not easily welded.

5-15. Deterioration: indicators and causes

Both wrought iron and cast iron are subject to the same causes of deterioration as structural steel (previously discussed). It should be noted that cast iron is subject to defects such as checks (cracking due to tensile cooling stresses) and blowholes. The latter has a serious effect on both the strength and toughness of the cast iron.

Section V. STONE MASONRY

5-16. General

Stone masonry is little used today, except as facing or ornamentation. However, some old stone structures are still in use. The types of stones commonly used in bridges are granite, limestone, and sandstone, although many smaller bridges or culverts were built of stones locally available.

5-17. Physical and mechanical properties

a. Strength. Stone has more than adequate strength for most loads.

b. Porosity. While all stone is porous, sandstone and some limestones are much more porous than granite.

c. Absorption. Most stone is absorptive, especially limestone.

d. Thermal expansions. Stone expands and contracts with temperature variations.

e. Thermal conductivity. Stone is generally a poor conductor of heat.

f. Durability. Stone is more durable than most materials, although there is a wide range in durability between different varieties of stone.

g. Fire resistance. While not flammable, stone can be damaged by fire.

5-18. Indicators of deterioration

The following terms should be used to describe deterioration of stone masonry structures. The description, extent, and location of the deterioration should be reported.

a. Weathering. The hard surface degenerates into small granules, giving stones a smooth, rounded look.

b. Spalling. Small pieces of rock break out or chip away.

c. Splitting. Seams or cracks open up in the rocks, eventually breaking them into smaller pieces.

5-19. Causes of deterioration

a. Chemical. Gasses and solids dissolved in water often attack rocks chemically. Some of these solutions can dissolve cementing compounds between the rocks. Oxidation and hydration of some compounds found in rock will also damage.

b. Seasonal expansion and contraction. Repeated volume changes produced by seasonal expansion and contraction will cause tiny seams to develop, thereby weakening the rock.

c. Frost and freezing. Water freezing in the seams and pores of rocks can split or spall rock.

d. Abrasion. Abrasions are due mostly to wind or waterborne particles.

e. Plant growth. Lichens and ivy will attack stone surfaces chemically in attaching themselves to the stone. Roots and stems growing in crevices or joints exert a wedging force.

f. Marine borers. Rock-boring mollusks attack rock by means of chemical secretions.

Section VI. ALUMINUM

5-20. General

a. While aluminum has been widely used for signs, light standards, railings, and sign bridges, it is seldom used as the principal material in the construction of vehicular bridges. While most properties of aluminum are similar to those of steel, the following differences exist:

(1) *Lightness.* Aluminum weighs about one-third (1/3) as much as steel.

(2) *Strength.* Aluminum, while not as strong as steel, will be made comparable to steel in strength when alloyed.

(3) *Corrosion resistance.* Aluminum is highly resistant to atmospheric corrosion.

(4) *Workability.* Aluminum is easily fabricated. However, welding requires special procedures.

(5) *Durability.* Aluminum is durable.

5-21. Deterioration: indicators and causes

a. Cracking. Aluminum may be subject to some fatigue cracking. Aluminum members should be examined in areas near the bases of cantilever arms and in areas near complex welded and riveted connection. Weld cracking often occurs on bridge signs due to stresses caused from the misalignment of prefabricated sections and due to fatigue caused from wind and vibration loadings.

b. Pitting. Aluminum will pit slightly, but this condition rarely becomes serious.

Section VII. FOUNDATION SOILS

5-22. General

a. Most foundation movements are caused by movement of the supporting soil. For this reason, it is desirable to give a brief description of these movements, although the basic theory involved is beyond the scope of this manual.

b. Soil deformations are caused by volume changes in the soil or by a shear failure. Slope slides and bearing failures are good examples of shear failures. Where loads are not large enough to cause shear failure, settlements may still occur as a result of volume change. The length of time and magnitude of the settlements depend upon the composition of the soil. Granular solids, such as sand, will usually undergo a relatively small volume change in a short period of time. However, cohesive soils such as clay can undergo large deformations or volume changes, which may continue for years. This latter process is called consolidation and is usually confined to clays and clayey silts.

c. Substructures that are supported directly by a cohesive soil may continue to settle for a long period of time. Consolidation usually produces vertical settlement.

5-23. Types of movement

For convenience, foundation movements may also be classified into the following categories:

a. Lateral movements. Earth-retaining structures, such as abutments and retaining walls, are susceptible to lateral movements, although piers sometimes also undergo such displacements.

b. Vertical movements (settlements). Any type of substructure not founded on solid rock may be subject to settlement.

c. Pile settlements. While pile settlement could be listed under lateral or vertical movements, it is mentioned separately since there is a tendency to consider piles as a panacea for all foundation problems. In addition, some of the causes of failure are peculiar to pile foundations.

d. Rotational movement (tipping). Rotation movement of substructures can be considered to be the result of unsymmetrical settlements or lateral movements. It will be discussed under the movement that is typical of the various substructures.

5-24. Effects on structures

The effects of foundation movements upon a structure will vary according to the following factors:

a. Magnitude of movements. All foundations undergo some settlement, even if only elastic compression of ledge or piles. All sizable footings probably will experience a minute differential settlement. However, very small foundation movements have no effect. Simple structures, and those with enough joints, will tolerate even moderate differential displacements with little difficulty other than minor cracking and the binding of end dams. Movements of large magnitudes, especially when differential, cause distress in nearly all structures (paragraph 5-25b(2)). Large movements will cause deck joints to jam; slabs to crack; bearings to shift; substructures to crack, rotate, or slide; and superstructures to crack, buckle, and possibly, even to collapse. The larger the settlement to be accommodated within a given distance, the more structural damage can be anticipated.

b. Type of settlement.

(1) *Uniform settlement.* A uniform settlement of all the foundations of a bridge will have little effect upon the structure. Settlements of nearly 1 foot have been experienced by small (70-foot), single-span bridges with no sign of appreciable distress.

(2) *Differential settlement.* Differential settlement can produce serious distress in any bridge. Where the differential settlement occurs between different substructure units, the magnitude of the damage depends on the bridge type and span length. Should a differential settlement take place beneath the footings of the same substructure, damage can vary from an opening of the vertical expansion joints between the wing wall and the abutment to severe tipping and cracking of walls or other members (See figures 5-19 through 5-21). Scour can cause support settlement (figure 5-22) or complete failure (figure 5-23).

c. Type of structure.

(1) *Simple (determinant).* As mentioned, the strength of a simple, or determinant, structure usually is not affected by movements unless they are quite large. There are usually enough joints to permit the movements without major damage to the basic integrity of the structure. At most, some finger joints or bearing may require resetting, or

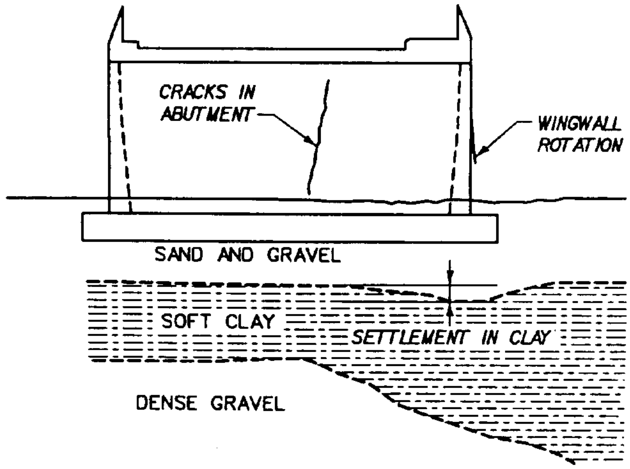

Figure 5-19. Differential settlement under an abutment.

Figure 5-20. Differential settlement.

beam supports may need shimming. However, pile bent or trestle bridges are very vulnerable since a large settlement or movement of a bent could cause the superstructure to fall off a narrow bridge seat, leading to the loss of the bridge spans.

(2) *Indeterminant.* An indeterminant bridge is seriously affected by differential movements, since such movements at supports will redistribute the loads, possibly causing large overstresses. For example, a fixed-end arch could be severely damaged if a foundation rotates. Most continuous bridges have fewer joints than simple-span bridges. These bridges are very likely to be damaged if subjected to displacements which are greater in magnitude, or different in direction, from those that were considered in the original designs.

5-25. Indicators of movement

Foundation movements may often be detected by first looking for deviations from the proper geometry of the bridge. With the exception of curved structures, haunched members, and steeply inclined bridges, members and lines should usually be parallel or perpendicular to each other. While not always practical, especially for bridges spanning large bodies of water or for those located in urban industrial areas, careful observation of the overall structure for lines that seem incongruous with the rest of the bridge is a good starting point. For a more detailed inspection, the following methods are often useful:

a. Check the alignment. Any abrupt change or kink in the alignment of the bridge may indicate a lateral movement of a pier or of bearings. Older bridges are particularly vulnerable to ice pressures which can cause structural misalignment.

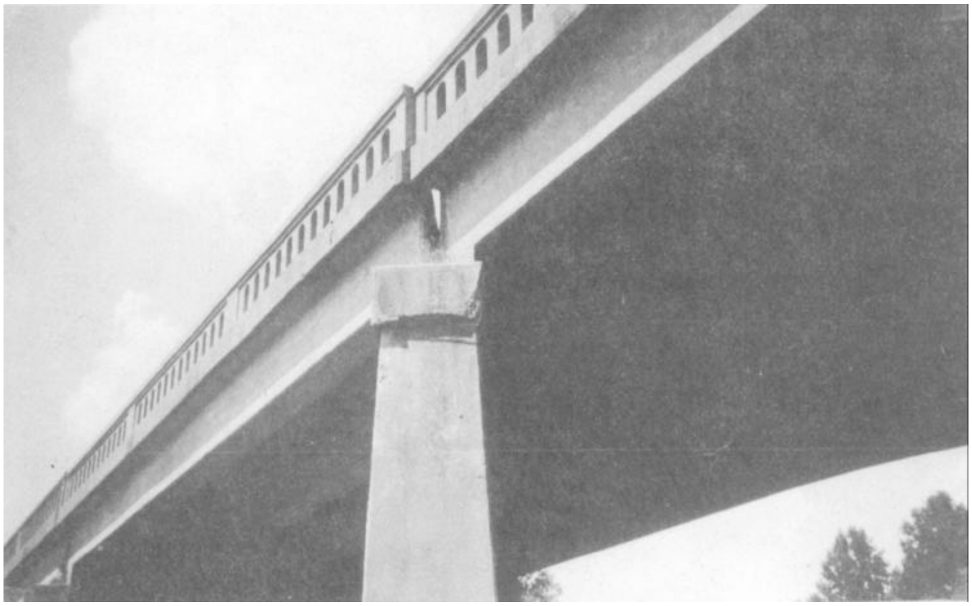

Figure 5-21. Differential pier movement causing superstructure movement.

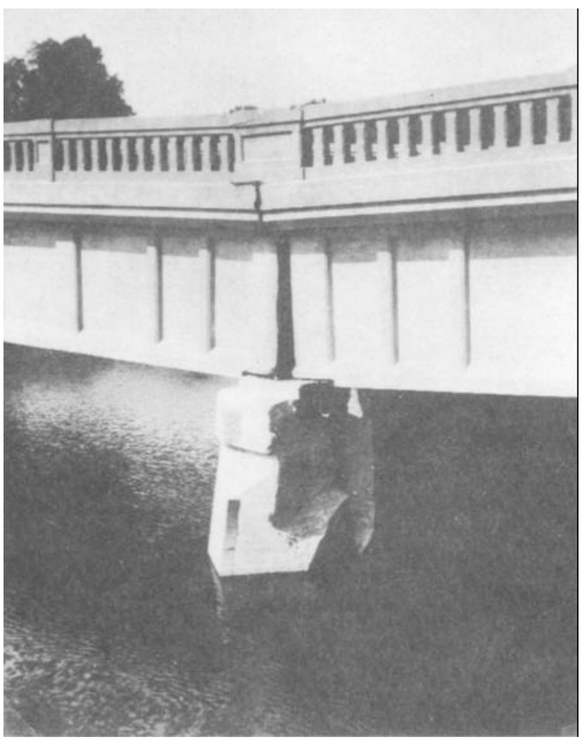

Figure 5-22. Movement due to scour.

b. Sight along railings. A sudden dip in the rail line is often the result of settlement of a pier or abutment.

c. Run profile levels along the centerline and/or the gutter lines. This inspection technique will not only help to establish the existence of any settlement but will also identify any differential settlements across the roadway. Normally, this kind of inspection technique will be employed only for large bridges or where information concerning the extent and character of differential settlement movement is required.

d. Check piers, pile bents, and abutment faces for plumbness with a transit. This inspection method provides an excellent check for the simpler techniques of plumbness determination. An out-of-plumb pier in either direction usually signifies foundation movement; it may also indicate a superstructure displacement. For small bridges and for preliminary checks, the use of a plumb bob is an adequate means for determining plumbness.

e. Observe the inclination of expansion rockers or roller movements. Rocker inclinations inconsistent with seasonal weather conditions may be a sign of foundation or superstructure movement. Of course, this condition may also indicate that the expansion rockers were set improperly. Out-of-plumb hangers on cantilevered structures are another indication of foundation shifting.

f. Observe expansion joints at abutments and walls. Observe the expansion joints for signs of opening or rotating. These conditions may indicate the movement of subsurface soils or a bearing failure under one of the footings.

g. Check deck joints and finger dams. Abnormally large or small openings, elevation differential, or jamming of the finger dams can be caused by substructure movements. Soil movements under the approach fills are also frequent occurrences.

h. Observe slabs, walls, and members. Cracks, buckling, and other serious distortions should be

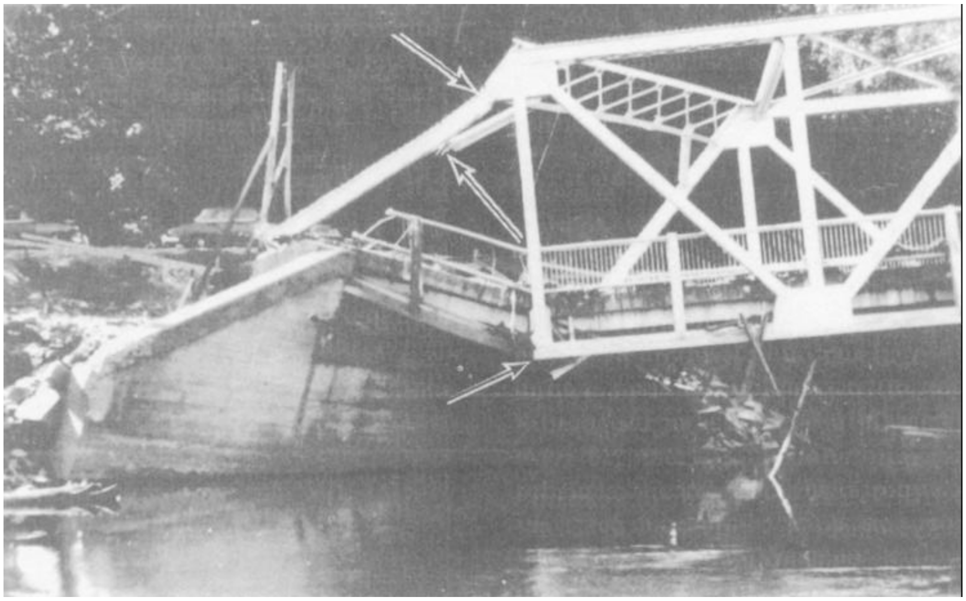

Figure 5-23. Abutment failure from scour.

noted. Bracing, as well as the main supporting sections, should be scrutinized for distortion.

i. Check backwalls and beam ends. Check the backwalls for cracking which may be caused by either abutment rotation, sliding, or pavement thrust. Check for beam ends which are bearing against the backwall. This condition is a sign of horizontal movement of the abutment.

j. Observe fill and excavation slopes. Slide scarps, fresh sloughs, and seepage are indications of past or imminent soil movement.

k. Scour. *See* "Waterways," section VIII of this chapter.

l. Unbalanced postconstruction embankment or fills. Embankments or fills should be checked for balance and positioning. Unbalanced embankments or fills can cause a variety of soil movements which may impair the structural integrity of the bridge.

m. Underwater investigation of all piling and pile bents. Underwater investigation of piling and pile bents should be undertaken periodically. Check all timber piles for insect attack and deterioration. Examine steel piles well below the water surface. Steel piles protected in the splash zone can rust between the concrete jacket and the mudline. Examine prestressed piles below water for cracking or splitting.

5-26. Causes of foundation movements

The following causes of foundation movements, except as specifically noted, can produce lateral and/or vertical movements depending on the characteristics of the loads or substructures:

a. Slope failure (embankment slides). These are shear failures manifested as lateral movements of hillsides, cut slopes, or embankments. Footing or embankment loads imposing shear stresses greater than the soil shear strength are common causes of slides (figure 5-24, part a).

b. Bearing failures. Bearing failures are settlements or rotations of footings due to a shear failure in the soil beneath (figure 5-24, part b). When bearing or slope failures take place on an older structure, it usually indicates a change in subsurface conditions. This may endanger the security of nearby structures and foundations.

c. Consolidation. Serious settlement can result from consolidation action in cohesive soils. Settlement of bridge foundations may be caused by changes in the groundwater conditions, the placement of additional embankments near the structure, or increases in the height of existing embankments.

d. Seepage. The flow of water from a point of higher head (elevation or pressure) through the soil to a point of lower head is seepage (figure 5-24, part c). Seepage develops a force which acts on the soil through which the water is passing. Seepage results in lateral movement of retaining walls by:

(1) An increase in weight (and lateral pressure) of the backfill because of full or partial saturation.

(2) A reduction in resistance provided by the soil in front of the structure.

e. Water table variations. Large cyclic variations in the elevation of the water table in loose granu-

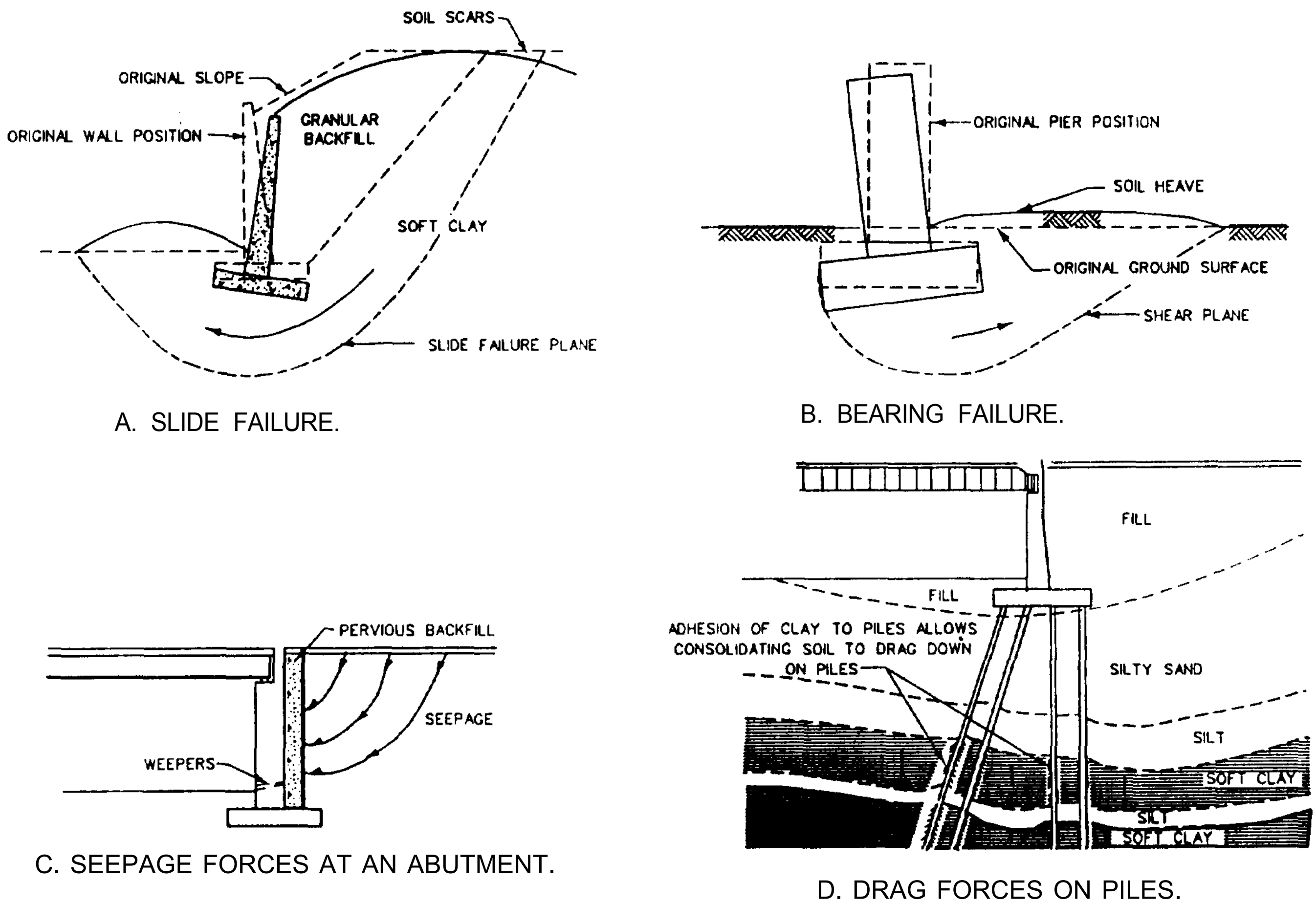

Figure 5-24. Causes of foundation movement.

lar soils may lead to a compaction of the upper strata. The effects of noncyclic changes in the water table such as consolidations, slides, and seepage were previously discussed. Changes in the water table may also change the characteristics of the soil which supports the foundation. Changes in soil characteristics may, in turn, result in the lateral movement or the settlement of the foundation.

f. Frost action. Frost heave in soil is caused by the growth of ice lenses between the soil particles. Footings located above the frost line may suffer from the effects of frost heave and a loss in bearing capacity due to the subsequent softening of the soil. The vertical elements on light trestle bents may also be lifted by frost and ice actions.

g. Expansive soils. Some clays, when wet, absorb water and expand, placing large horizontal pressures on any wall retaining such soil. Structures founded on expansive clay may also experience vertical soil movements (reverse settlement).

h. Ice. Ice can cause lateral movement in two ways. Where fine-grained backfill is used in retaining structures and the water table is above the frost line, the expansion of freezing water will exert a very large force against a wall. The piers of river bridges are also subject to tremendous lateral loads when an ice jam occurs at the bridge.

i. Thermal forces from superstructures. On structures without expansion bearings, or where the expansion bearings fail to operate, thermal forces may tip the substructure units. Pavement thrust is another force that will have the same effect.

j. Drag forces. Additional embankment loads or very slow consolidation of a subsurface compressible stratum will exert vertical drag forces on the bearing piles which are driven through such material. This may cause yielding or failure of the piles (figure 5-24, part d).

k. Deterioration, insect attacks, and construction defects. All piles may develop weaknesses leading to foundation settlements from one or more of these causes:

(1) Timber, steel, and concrete piles are subject to loss of section because of decay, rusting, and deterioration.

(2) Timber piles are vulnerable to marine borers and ship worms.

(3) Construction defects include overdriven piles, underdriven piles, failure to fill pile shells

completely with concrete, or imperfect casings of a cast-in-place pile. Any of these defects will produce a weaker pile. Settlement will probably be gradual in improperly driven piles or in piles with weak or voided concrete. Piles suffering severe loss of section due to rust, spalling, chemical action, or insect infestation may fail suddenly under an unusually heavy load.

l. Scour and erosion. Scour can cause extensive settlement and/or structural failure as previously shown in figures 5-22 and 5-23. Since water will carry off particles of soil in suspension, a considerable hole can be formed around piers or other similar structural objects. This condition results in a greater turbulence of water and an increased size of soil particles that can be displaced. Scour is a very important consideration and will be given considerable attention in section VIII of this chapter. Erosion of embankments due to improper drainage (figure 5-25) can also lead to approach and abutment settlements.

m. Earth or rock embankments (stockpiles). Post-construction placement of embankments may cause instability since it will produce greater loads than were included in the original design.

Figure 5-25. Embankment erosion due to improper drainage. (Sheet 1 of 2)

Figure 5-25. Embankment erosion due to improper drainage. (Sheet 2 of 2)

Section VIII. WATERWAYS

5-27. General

A typical flow profile through a bridge is shown in figure 5-26. Note that the presence of a bridge (obstruction) does significantly affect the flow. Waterways should be inspected to determine whether any condition exists that could cause damage to the bridge or to the area surrounding the bridge. In addition to inspecting the channel's present condition, a record should be made of significant changes that have taken place in the channel, attributable to natural or artificial causes. When significant changes have occurred, an investigation must be made into the probable or potential effects on the bridge structure. Events which tend to produce local scour, channel degradation, or bank erosion are of primary importance.

5-28. Types of movement and effects on waterways

a. Scour. Scour is defined as the removal and transportation of material from the bed and banks of rivers and streams as a result of the erosive action of running water. Some general scouring (figure 5-27) takes place in all streambeds, particularly at flood stage. The characteristics of the channel influence the amount and nature of scour. Accelerated local scouring (figure 5-28) occurs where there is an interference with the streamflow, e.g., approach embankments extended in the river or piers and abutments constructed on the river bottom. The amount of scour in such cases depends on the degree to which streamflow is disturbed by the bridge and on the susceptibility of river bottom to scour action. Scour depth may range from zero in hard rock to 30 feet or more in very unstable river bottoms. In determining the depth of local scour, it is necessary to differentiate between true scour and apparent scour. As the water level subsides after flooding, the scour holes that are produced tend to refill with sediment. Elevations taken of the streambed at this time will not usually reveal true scour depth. However, since material borne and deposited by water will usually be somewhat different in character from the material in the substrate, it is often possible to determine the scour depth on this basis. If, for example, a strata of loose sand is found overlying a hard till substrate, it is reasonable to assume that the scour extends down to the depth of the till. This can often be confirmed by sounding or probing, provided the scour depth is limited to a few feet. Where coarse deposits or clays are encountered, sounding will probably be unsuccessful. Scour problems should be remedied as soon as possible since every flood can destroy the bridge totally. Typical situations which tend to lead to scour problems are as follows:

(1) *Sediment deposits.* The construction of an upstream dam, as seen in figure 5-29, will cause sediment previously carried downstream to be deposited in the reservoir, which acts as a settling basin. The increased scour capability of the downstream flow may degrade the lower channel.

(2) *Pier scour.* Scour around piers (figure 5-30) is greatly influenced by the shape of the pier and its skew to the direction of the flood flow. Note that the direction of flood flow will often be different from that of normal channel flow.

(3) *Loose riprap.* Loose riprap (figure 5-31) piled around piers to prevent local scour at the pier may cause deep scour holes to form downstream.

(4) *Lined banks.* Lined banks (figure 5-32) tend to reduce scour, but such a constriction might increase general scour in the bridge opening, especially at an adjacent or end pier.

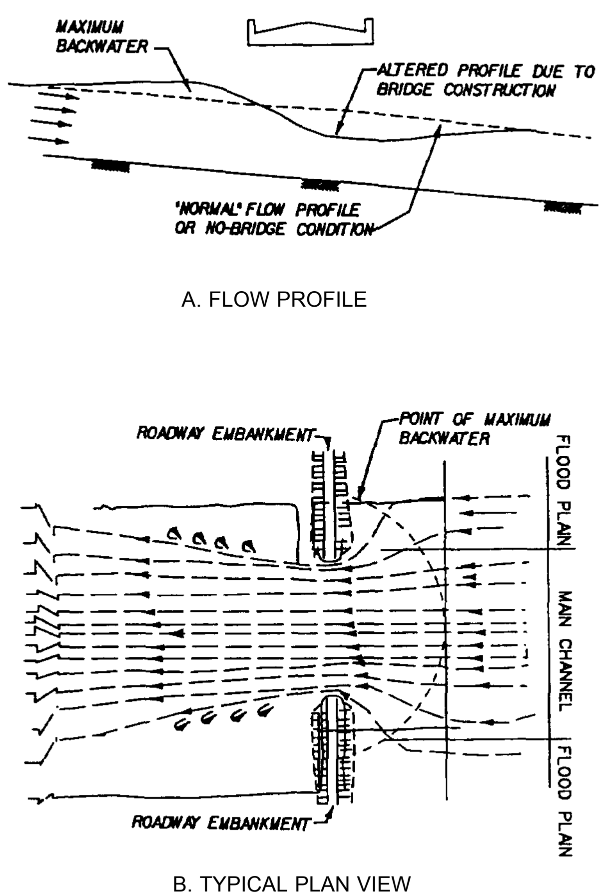

Figure 5-26. Typical flow characteristics through a bridge.

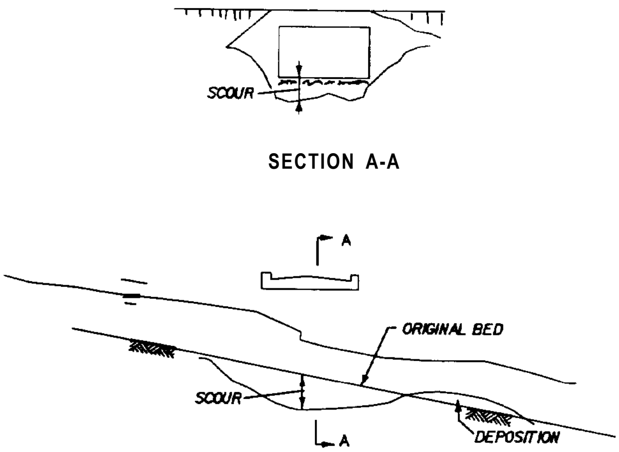

Figure 5-27. General scour

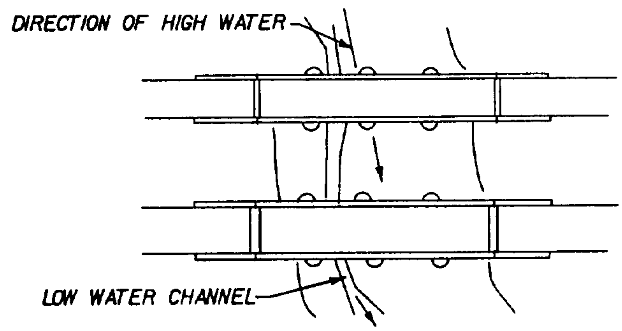

Figure 5-30. Pier scour.

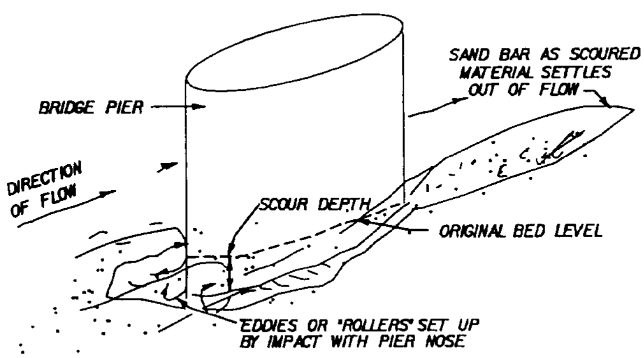

Figure 5-28. Localized scour.

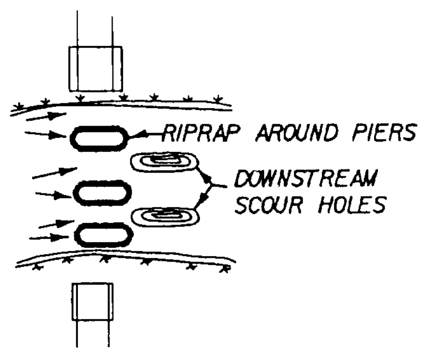

Figure 5-31. Loose riprap.

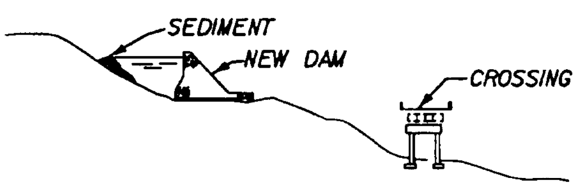

Figure 5-29. Sediment deposits.

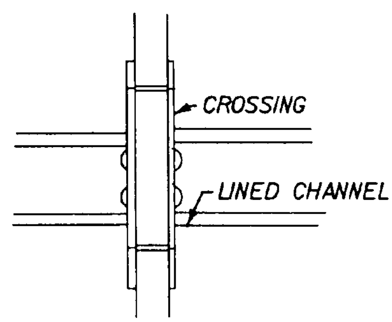

Figure 5-32. Lined banks.

(5) *Horizontal of vertical channel constrictions.* A firm or riprapped bottom or a horizontal constriction can cause a deep scour hole downstream with severe bank erosion resulting in downstream ponding as shown in figure 5-33.

(6) *Flooding.* During flood (figure 5-34) the waterway constriction may produce general scouring in the vicinity of the bridge.

(7) *Protruding abutments.* Protruding abutments (figure 5-35) may produce local scour. Deepest scour usually takes place at the upstream corner. The severity of scour increases with increased constriction.

(8) *Debris.* Collection of debris around piers (figure 5-36), in effect, enlarges the size of the pier and causes increased area and depth of scour.

(9) *River bends.* As seen in figure 5-37, a high scour potential exists for bridges located in the bend of the channel.

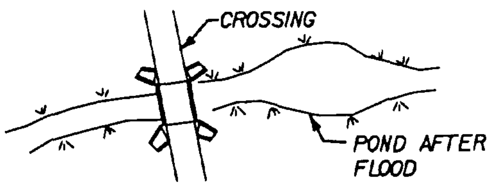

Figure 5-33. Channel constrictions.

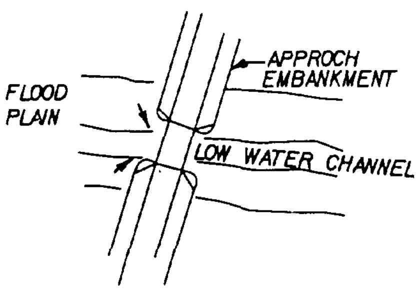

Figure 5-34. Flooding.

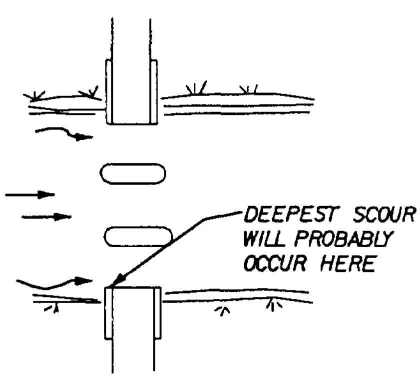

Figure 5-35. Protruding abutments.

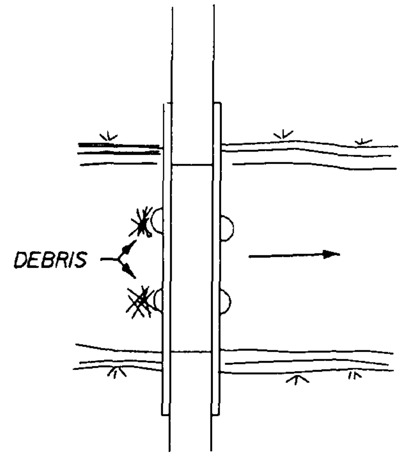

Figure 5-36. Debris problems.

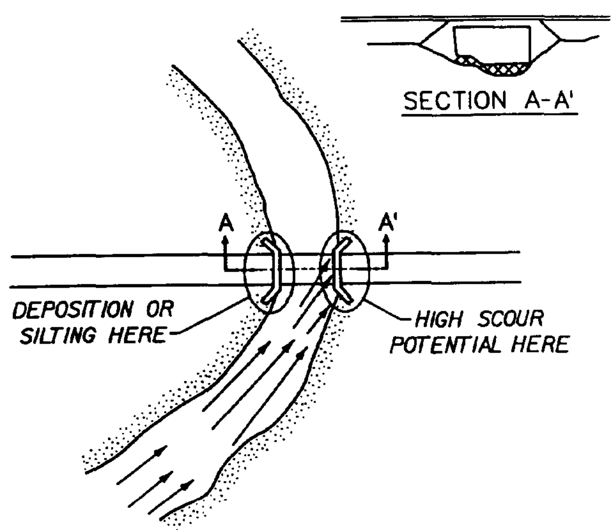

Figure 5-37. Bridge in a river bend.

b. Channel of streambed degradation. Streambed degradation is usually due to artificial or natural alteration in the width, alignment, or profile of the channel, upsetting the equilibrium or regime of the channel. These alterations may take place at the bridge site or some distance upstream or downstream. A channel is in regime if the rate of flow is such that it neither picks up material from the bed nor deposits it. In the course of years, the channel will gradually readjust itself to the changed condition and will tend to return to a regime condition. Streambed degradation and scour seriously endanger bridges with foundations located in erodible riverbed deposits and where the foundation does not extend to a depth below that of anticipated scour. Removal of material adjacent to the foundation may produce lateral slope instability causing damage to the bridge. Concrete slope protection (figure 5-38) or riprap (figure

5-39) is often provided to prevent bank erosion or to streamline the flow. It is particularly important where flow velocities are higher or where considerable turbulence is likely. It may also be necessary where there is a change in direction of the waterway. Slope cones around abutments are very susceptible to erosion and are usually protected. Situations which lead to channel degradation are as follows:

(1) *Channel change.* Changes in the channel (figure 5-40) steepen the channel profile and increase flow velocity. The entire upstream reach may degrade.

(2) *Removal of material.* Removal of large quantities of material (figure 5-41) (such as by dredging or gravel borrow pits) from the downstream channel will cause increased upstream flow velocities and thus degradation.

(3) *Removal of obstruction.* A downstream obstruction (figure 5-42) will cause the flow under the bridge to be deep and slow. Once the obstruction is removed, the flow becomes more shallow and more rapid, causing degradation.

c. Waterway adequacy. Scour and streambed degradation are actually the result of inadequate waterway areas (freeboard). The geometry of the channel, the amount of debris carried during high water periods, and the adequacy of freeboard should be considered in determining waterway adequacy. Where large quantities of debris and ice are expected, sufficient freeboard is of the greatest importance.

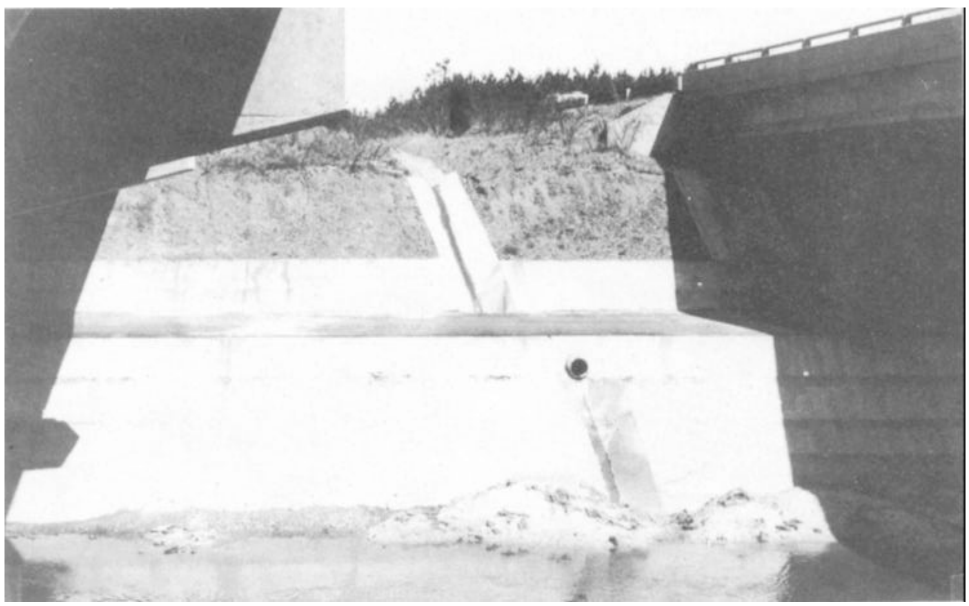

Figure 5-38. Concrete slope protection.

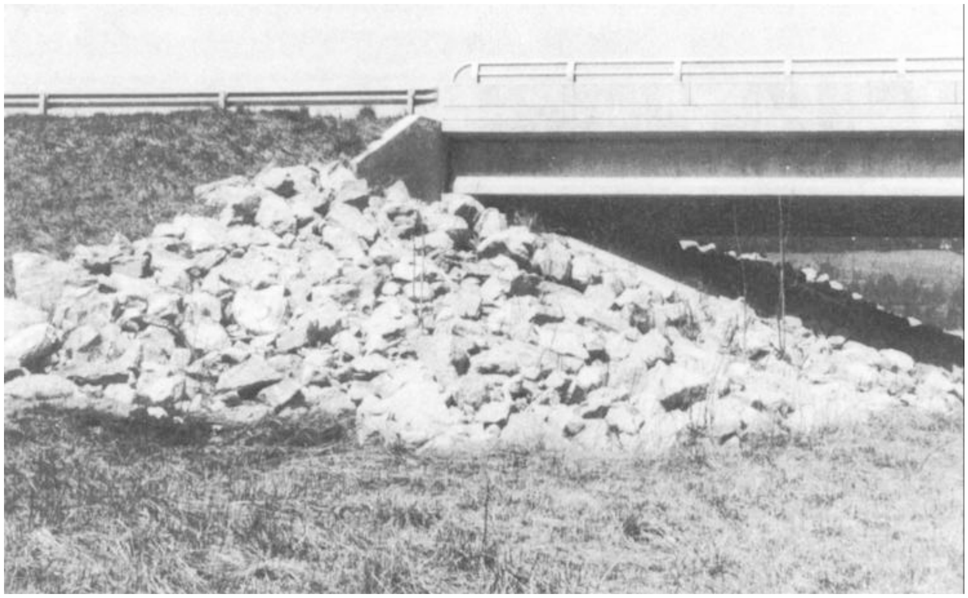

Figure 5-39. Riprap slope protection.

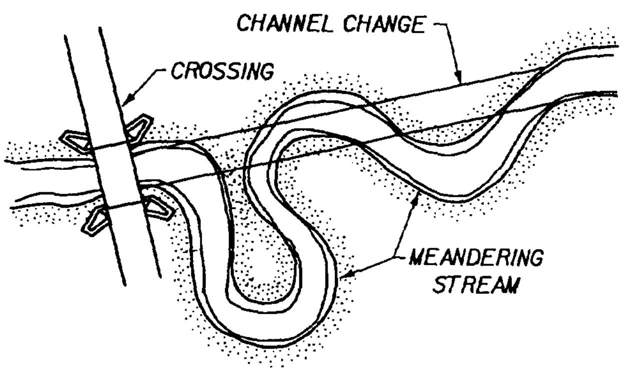

Figure 5-40. Channel change.

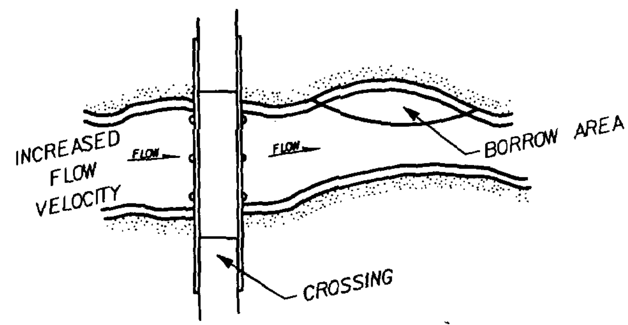

Figure 5-41. Material removal.

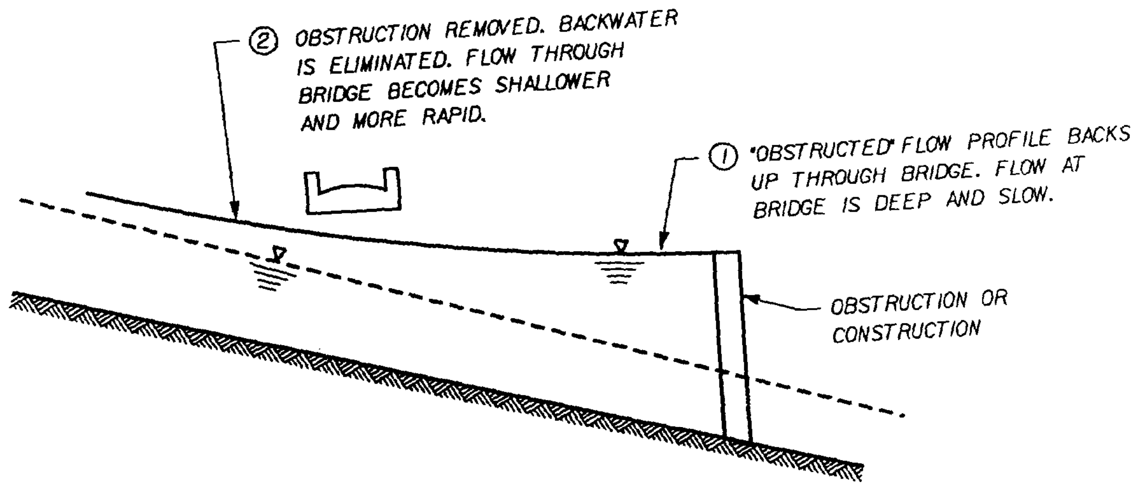

Figure 5-42. Obstruction removal.

APPENDIX A

REFERENCES AND BIBLIOGRAPHY

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Government Publications

Departments of the Army, Navy, and Air Force

AR 420-72	Surfaced Areas, Bridges, Railroad Track and Associated Appurtenances
FM 5-446	Military Nonstandard Fixed Bridging
TM 5-628	Railroad Track Standards
FM 5-277	Bailey Bridge

Coast Guard

Pamphlet CG204	Aids to Navigation

Department of Transportation, Federal Highway Administration

Bridge Inspector's Training Manual 70
Inspection of Fracture Critical Bridge Members

Nongovernment Publications

American Association of State Highway Transportation Officials (AASHTO), 444 North Capitol Street, Washington, DC 20001

Standard Specifications for Highway Bridges, Fourteenth Edition, 1989
Manual on Uniform Traffic Control Devices
Manual for Maintenance Inspection of Bridges

National Cooperative Highway Research Program (NCHRP), Transportation Research Board, National Research Council, Washington, DC

Report 280	Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members, December 1985
Report 293	Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members, November 1980

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Annual Book of American Society for Testing and Materials (ASTM) Specification, 1990. American Society for Testing and Materials, 1961 Race Street, Philadelphia, PA.

Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, 1989. American Association of State Highway and Transportation Officials, Washington, DC (1989).

Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members, 1978. American Association of State Highway and Transportation Officials, Washington, DC (1989).

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Manual for Bridge Maintenance Planning and Repair Methods, Volume I. Florida Department of Transportation, State Maintenance Office.

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FM 5-551	Carpentry
TM 5-744	Structural Steel Work
TM 5-622/MO-104/AFM 91-34	Maintenance of Waterfront Facilities

Reports

National Cooperative Highway Research Program, Report 293. “Methods of Strengthening Existing Highway Bridges.” Transportation Research Board, National Research Council, Washington, DC, September 1987.

National Cooperative Highway Research Program, Report 333. “Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges.” Transportation Research Board, National Research Council, Washington, DC, December 1990.

Journal Articles

American Concrete Institute Committee 201, “Guide to Durable Concrete,” *Journal of the American Concrete Institute*, Number 12, Proceedings Volume 74, pages 573-609, December 1977.

APPENDIX B

SUGGESTED ITEMS FOR ARMY ANNUAL AND AIR FORCE BIANNUAL BRIDGE INSPECTIONS

BRIDGE INSPECTION ITEMS

Include the following items:
1. Installation.
2. Bridge number.
3. Location.
4. Date inspected.
5. Existing bridge classification (if applicable).

For the following components, address each appropriate inspection item and make notes of any observed deficiencies and recommendations:

A. Timber Abutments
1. Signs of settlement.
2. Rusting of steel rods.
3. Decay of end dam, wingpost, post, and/or cap.
4. Deterioration of block (bearing and anchor).
5. Decay of sill and footing.
6. Loose timbers.
7. Decay of breakage of piles (wing or bearing).

B. Steel Pile Abutments
1. Settlement.
2. Rusting of end dam, pile and/or cap.
3. Section loss of steel members.
4. Missing, loose, or rusting bolts.

C. Concrete Abutments, Wingwalls, and Retaining Walls
1. Settlement.
2. Proper function of weep holes.
3. Cracking or spalling of bearing seats.
4. Deterioration of cracking of concrete.
5. Exposed reinforcing steel.

D. Timber Piers and Bents
1. Settlement.
2. Decay of caps, bracing, scabbing, or corbels.
3. Missing posts or piles.
4. Decay of posts or piles.
5. Debris around or against piers.
6. Section loss of sills or footings.
7. Erosion around piers.
8. Rusting of wire-rope cross bracing.
9. Loose or missing bolts.
10. Splitting or crushing of the timber when:
    a. The cap bears directly upon the cap, or
    b. Beam bears directly upon the cap.
11. Excessive deflection or movement of members.

E. Steel Piers and Bents
1. Settlement or misalignment.
2. Rusting of steel members or bearings.

3. Debris.
4. Rotation of steel cap due to eccentric connection.
5. Braces with broken connections or loose rivets or bolts.
6. Member damage from collision.
7. Need for painting.
8. Signs of excessive deflection or movement of members.

F. Concrete Piers and Bents

1. Settlement.
2. Deterioration or spalling of concrete.
3. Cracking of pier columns and/or pier caps.
4. Cracking or spalling of bearing seats.
5. Exposed reinforcing steel.
6. Debris around piers or bents.
7. Section loss of footings.
8. Erosion around piers.
9. Collision damage.

G. Concrete (girders, beams, frames, etc.)

1. Spalling (give special attention to points of bearing).
2. Diagonal cracking, especially near supports.
3. Vertical cracks or disintegration of concrete, especially in the area of the tension steel.
4. Excessive vibration or deflection during vehicle passage.
5. Corrosion or exposure of reinforcing steel.
6. Corroded, misaligned, frozen, or loose metal bearings.
7. Tearing, splitting, bulging of elastomeric bearing pads.

H. Timber (trusses, beams, stringers, etc.)

1. Broken, deteriorated, or loose shear connectors.
2. Failure, bowing, or joint separation of individual members of trusses.
3. Loose, broken, or worn planks on the timber deck.
4. Improper functioning of members.
5. Rotting or deterioration of members.

I. Steel (girders, stringers, floor beams, diaphragms, cross frames, portals, sway frames, lateral bracing, truss members, bearing and anchorage, eyebars, cables, and fittings)

1. Corrosion and deterioration along:
   a. Web flange.
   b. Around bolts and rivets heads.
   c. Under deck joints.
   d. Any other points which may be exposed to roadway drainage.
   e. Eyebars, cables, and fittings.
2. Signs of misalignment or distortion due to overstress, collision, or fire.
3. Wrinkles, waves, cracks, or damage in the web and flange of steel beam, particularly near points of bearing.
4. Unusual vibration or excessive deflection occurring during the passage of heavy loads.
5. Frozen or loose bearings.
6. Splitting, tearing, or bulging in elastomeric bearing pads.

J. Concrete Appurtenances

1. Cracking, scaling, and spalling on the:
   a. Deck surface.
   b. Deck underside.
   c. Wearing surface (map cracking, potholes, etc.).

NOTE: If deterioration is suspected, remove a small section of the wearing surface in order to check the condition of the concrete deck.

2. Exposed and/or rusting reinforcing steel.
3. Loose or deteriorated joint sealant.

4. Adequacy of sidewalk drainage.
5. Effect of additional wearing surfaces on adequacy of curb height.

K. Timber Appurtenances

1. Loose, broken, or worn planks.
2. Evidence of decay, particularly at the contact point with the stringer where moisture accumulates.
3. Excessive deflection or loose members with the passing of traffic.
4. Effect of additional wearing surfaces on adequacy of curb height.

L. Steel Appurtenances (including but not limited to decks, gratings, curbs, and sidewalks)

1. Corroded or cracked welds.
2. Slipperiness when deck or steel sidewalk is wet.
3. Loose fasteners or loose connections.
4. Horizontal and vertical misalignment and/or collision damage.

M. Masonry Bridges

1. Settlement.
2. Proper function of weep holes.
3. Collision damage.
4. Spalling or splitting of rocks.
5. Loose or cracked mortar.
6. Plant growth, such as lichens and ivy, attaching to stone surfaces.
7. Marine borers attacking the rock and mortar.

N. Miscellaneous

1. Existence and appropriateness of bridge classification signs.
2. Condition of approachments.
3. Leaks, breaks, cracks, or deterioration of pipes, ducts, or other utilities.
4. Damaged or loose utility supports.
5. Wear or deterioration in the shielding and insulation of power cables.

APPENDIX C

SUGGESTED ITEMS FOR ARMY TRIENNIAL AND EVERY THIRD AIR FORCE BIANNUAL BRIDGE INSPECTIONS

BRIDGE INSPECTION ITEMS

Include the following items:

A. General Information to Include

1. Bridge name.
2. Location.
3. Date of inspection.
4. Design load (if known).
5. Military load classification (if known).
6. Date built.
7. Traffic lanes.
8. Transverse section (describe or sketch).
9. Structure length.
10. No. of spans.
11. Plans available.
12. Inspection records.
    a. Year inspected.
    b. Inspector.
    c. Qualification.
13. Bridge description.
    a. Floor system.
    b. Beams.
    c. Girders.
    d. Stringers.
    e. Trusses.
    f. Suspension.
    g. Piers.
    h. Abutment A.
    i. Abutment B.
    j. Foundation.
    k. Piers or bents.
        (1) Caps.
        (2) Posts or columns.
        (3) Footings.
        (4) Piles.
        (5) Other.
    l. Deck:
        (1) Wearing surface.
        (2) Curb.
        (3) Railings.
        (4) Sidewalk.
        (5) Other.

B. Bridge Components Rating Information

The following items may be rated using the suggested ratings from part C of this appendix. Descriptive remarks may also be included.

1. Traffic safety features.
    a. Bridge railing.

b. Transitions.
c. Approach guardrail.
d. Approach guardrail terminal.
2. Deck.
a. Wearing surface.
b. Deck structural condition.
c. Curbs.
d. Median.
e. Sidewalk.
f. Parapet.
g. Railings.
h. Drains.
i. Lighting.
j. Utilities.
k. Expansion joints.
3. Load bearing components.
a. Bearing devices.
b. Stringers.
c. Girders or beams.
(1) General.
(2) Cross frames.
(3) Bracing.
d. Floor beams.
e. Trusses.
(1) General.
(2) Portals.
(3) Bracing.
f. Paint.
4. Abutments.
a. Wings.
b. Backwall.
c. Bearing seats.
d. Breast wall.
e. Weep holes.
f. Footing.
g. Piles.
h. Bracing.
i. Erosion or scour.
j. Settlement.
5. Piers/bents or pile bents.
a. Caps.
b. Bearing seats.
c. Column, stem, or wall.
d. Footing.
e. Piles.
f. Bracing.
g. Erosion or scour.
h. Settlement.
6. Channel and channel protection.
a. Channel scour.
b. Embankment erosion.
c. Drift.
d. Vegetation.
e. Fender system.
f. Spur dikes and jetties.

g. Rip rap.
h. Adequacy of opening.
7. Approach.
a. Alignment.
b. Approach.
c. Relief joints.
d. Approach.
(1) Guardrail.
(2) Pavement.
(3) Embankment.

C. Suggested Component Ratings

1. Traffic Safety Features.

Code	*Description*
0	Inspected feature DOES NOT currently meet acceptable standards or a safety feature is required and NONE IS PROVIDED.
1	Inspected feature MEETS currently acceptable standards.
N	NOT APPLICABLE

2. Superstructure, Substructure, Channel and Channel Protection, and Approach.

Code	*Description*
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION-no problems noted.
7	GOOD CONDITION-some minor problems.
6	SATISFACTORY CONDITION-structural elements show some minor deterioration.
5	FAIR CONDITION-all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION-advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION-loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION-advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION-major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION-out of service-beyond corrective action.

3. Supplemental for Channel and Channel Protection (Use in conjunction with part 2 above).

Code	*Description*
N	NOT APPLICABLE bridge is not over a waterway.
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.
7	Bank protection is in need of minor repairs. River control devices and embankment protection have little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the waterway slightly.
5	Bank protection is being eroded. River control devices or embankment have major damage. Trees and brush restrict the channel.

Code	*Description*
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the waterways.
3	Bank protection has failed. River control devices have been destroyed. Stream bed aggradation, degradation, or lateral movement has changed the waterway to now threaten the bridge or approach roadway.
2	The waterway has changed to the extent the bridge is near a state of collapse.
1	Bridge is closed because of channel failure. Corrective action may put it back in light service.
0	Bridge is closed because of channel failure. Replacement is necessary.

4. Supplemental for Approach Roadway Alignment (Use in conjunction with part 2 above):

Code	*Description*
8	Speed reduction is NOT required.
6	A VERY MINOR speed reduction is required.
3	A SUBSTANTIAL speed reduction is required.

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Bridge Inspection, Maintenance and Repair Part II
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1. Which is **NOT** a type of force exerted on members?

    a) tension
    b) shear
    c) compression
    d) all of the above

2. Shear stresses on a concrete beam causes __________ cracks.

    a) vertical
    b) horizontal
    c) diagonal
    d) no

3. Concrete has good tensile strength.

    a) True
    b) False

4. Isolated cracks indicate __________.

    a) over bending in a beam
    b) tension in the direction normal to the crack
    c) both a) and b)
    d) none of the above

5. __________ is the gradual and continuing loss of surface mortar and aggregate over an area.

    a) Scaling
    b) Disintegration
    c) Over cracking
    d) Spalling

6. __________ is defined as the development of fragments. Usually in the shape of flakes, detached from a larger mass.

    a) Scaling
    b) Disintegration
    c) Over cracking
    d) Spalling

7. Causes of deterioration include __________.

    a) foundation movements
    b) shrinkage
    c) corrosion of embedded metals
    d) all of the above

8. __________ are used in the evaluation of concrete systems.

    a) Core drilling
    b) Laboratory investigations
    c) Nondestructive testing
    d) all of the above

9. Indicators of deterioration in steel members include all, **EXCEPT** __________.

    a) rust
    b) cracking
    c) buckles and kinks
    d) spalling

10. Causes of deterioration for steel members include __________.

    a) fatigue and stress concentrations
    b) deicing agents
    c) thermal strains or overloads
    d) all of the above

11. Some of the nondestructive testing methods include __________.

    a) dye penetrant
    b) magnetic particle testing
    c) radiographic inspection
    d) all of the above

12. Under certain conditions and when properly treated or protected, timber is quite durable. However, timber is not a particularly durable material under all conditions. It should be noted that some preservative treatments reduce the __________ of timber.

    a) flammability resistance
    b) ductility
    c) strength
    d) fatigue resistance

13. Some of the deterioration causes of timber include all of the following, **EXCEPT** __________.

    a) fungus
    b) vermin
    c) weathering and warping
    d) pressure treatment

14. The fibrous nature of wrought iron produces a tight rust which is __________ likely to progress to flaking and scalling than is rust on carbon steel.

    a) less
    b) more
    c) the same
    d) does not affect wrought iron

15. Due to its high carbon content, cast iron is not easily welded.

    a) True
    b) False

16. __________ occurs when seams or cracks open up in the rocks, eventually breaking them into smaller pieces.

    a) Chemical cracking
    b) Weathering
    c) Spalling
    d) Splitting

17. Earth-retaining structures, such as abutments and retaining walls, are susceptible to __________ movements, although piers sometimes also undergo such displacements.

    a) rotational
    b) vertical
    c) lateral
    d) both a) and c)

18. Foundation movements may often be detected by __________.

    a) checking the alignment
    b) sight along railings
    c) running profile levels along the centerline
    d) all of the above

19. __________ is defined as the removal and transportation of material from the bed and banks of rivers and streams as a result of the erosion.

   a) Scour
   b) Seepage
   c) Settlement
   d) all of the above

20. Typical situations which tend to lead to scour problems include all, **EXCEPT** __________.

   a) sediment deposits
   b) pier scour
   c) loose riprap
   d) droughts