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Structural Health Monitoring Technologies for Concrete Structures—Report

Reported by ACI Committee 444

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Structural Health Monitoring Technologies for Concrete Structures—Report

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Structural Health Monitoring Technologies for Concrete Structures—Report

Reported by ACI Committee 444

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This report gives an overview of structural health monitoring (SHM) technologies for concrete structures. Data processing, analysis, and interpretation are not addressed in this document.

Keywords: sensors; structural health monitoring (SHM); structures; technology.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

The objective of this report is to introduce structural health monitoring (SHM) technologies and their applications to concrete structures. SHM entails the use of instruments and sensors to monitor changes of structural performance. Monitoring structural performance may ensure proper functioning and safety during the service life of a structural member or system. A wide variety of technologies are used to monitor the response of a structure. The response data, together with other data such as loading, environmental conditions, and other inputs, provide more complete information about the structural performance in terms of structural behavior or condition. Because these technologies have different operating principles, measure different quantities, and exhibit different strengths and limitations, the user must be informed how each technology can and should be used to maximize the value of an SHM system. The information gathered from an SHM system may effectively support rational decision-making processes regarding maintenance and repair of an existing structure.

1.2—Scope

The scope of this report is to provide an overview of SHM as applied to plain, reinforced, and prestressed concrete structures; introduce the physical phenomena that may be monitored and for what purpose; and provide a detailed discussion of established and emerging SHM technologies. This report does not discuss specific methods of data collection, storage, transmission, filtering, analysis, and interpretation. The intent of this report is to inform engineers, owners, and other SHM technology users about available sensor technologies, including the physical principle upon which each is based; equipment and sensor descriptions; method(s) of deployment; strengths and limitations of each technology; summary of availability and degree of acceptance in practice; and to identify relevant guidance or standards for use where they exist. [Section 3.1](#) contains a discussion on the purpose and role of SHM, a broad overview on designing

an SHM system, and general considerations for selecting SHM technologies. A table is provided in [Section 3.1.3](#) to guide the reader to the appropriate technology sections in the document ([Chapter 4](#)). [Section 3.2](#) presents the fundamental behavior (nonlinearity, non-homogeneity, cracking, and time dependency) of concrete structures to assist the reader in understanding the specific challenges associated with unreinforced, reinforced, and prestressed concrete structures in the selection of sensors and the implementation of SHM systems. The various SHM technologies are discussed in Chapters 4 through 6. Each chapter documents the working principle, necessary equipment, method of deployment, strengths and limitations, readiness for field application, and applicable codes and standards for a single technology or systems of related technologies. Chapter 4 presents technologies that measure the structural response, including acceleration, displacement, strain, or rotation. [Chapter 5](#) presents technologies that measure inputs or stimuli that result in or affect a structural response such as load, environmental conditions such as temperature or humidity, or chemical activity. Emerging technologies, including conductive surface sensors and fiber-optic sensors, as well as related SHM systems such as energy harvesting, micro-electromechanical, and wireless sensor network systems are discussed in Chapter 6.

CHAPTER 2—DEFINITIONS

Please refer to the latest version of ACI Concrete Terminology for a comprehensive list of definitions. Definitions provided herein complement that resource.

acceleration—the rate of change of velocity with respect to time of a vibrating structural member.

accelerometer—sensor that may be mounted on a structure to measure acceleration at a point.

acoustic emission—stress wave resulting from a sudden, irreversible, and not repeatable release of strain from internal sources such as fracture due to externally-applied or thermally-induced loading.

acoustic emission event—single occurrence of an acoustic emission source, which must be recorded by multiple sensors for it to be associated with a specific source.

acoustic emission hit—count of the acoustic emissions exceeding a specified threshold, as recorded on an individual sensor.

aperture, radar—effective area or receiving cross section, as a measure of how effective an antenna is at receiving the power of electromagnetic radiation (such as radio waves).

aperture, ultrasonic transducer—diameter of sensing element; determines focusing angle of transducer.

autonomous enforcement—enforcing legal weight limits for illegal overweight trucks by issuing autonomous citations based on an advanced weigh-in-motion (WIM) system with a specified accuracy of gross vehicle weight.

b-value analysis—data analysis based on the slope of the cumulative-frequency (log-scale) versus acoustic emission amplitude plot.

bending plate, weigh-in-motion—high-strength steel plate instrumented with high-precision strain gauges at its

bottom and secured in a foundation frame within a pavement to weigh trucks at highway speed.

broadband—property of a sensor that is sensitive over a broad range of frequencies.

carbon nanotube—cylindrical nanoscale particle used in material science to modify mechanical and electrical properties of another material.

convolution—mathematical operation describing how one function operates on another function to produce a third function, used to describe how an input (for example, surface vibration) is changed by a system (for example, transducer) to produce an output (transducer output voltage).

data acquisition system—electronic system used to sample (or digitize) and store data gathered from a sensor.

Doppler shift—apparent change in frequency and wavelength of a wave observed by a receiver that is moving in relation to the wave source.

electrical resistance—opposition to the flow of electrons; inverse of electrical conductance.

fiber optic—glass fibers used to transmit light.

frequency response function—system impulse response function in the frequency domain fully describing the properties of a system such as a transducer, filter, amplifier, or data acquisition system.

geophone—passive sensor that may be mounted to the ground or a structure to measure movement (velocity) at a point; most commonly used for stress wave and seismic applications.

Green's function—transfer function describing relationship between unit impulse force at a particular location on a solid and the resulting response at another location.

high fidelity—of high quality, without distortion or bias.

inductive loop—setup that measures the fluctuations of current passed through a conductive loop or coil to detect the presence of ferromagnetic objects that disturb the resulting electromagnetic field.

interferogram—two-dimensional map representation of differences in phase values.

interferometry—the superimposing of two waves (for example, light or electromagnetic waves) to obtain additional information about the similarities and differences between the waves, which are typically measured from two adjacent, closely-spaced positions.

laser Doppler vibrometer—device that measures contactless measurements of the vibrations of a surface.

lead zirconate titanate—piezoelectric material typically used in transducer element to capture minute motions.

modal properties—natural vibration frequencies and mode shapes of a structural member or system.

moment tensor analysis—quantitative approach used to characterize the nature of an acoustic emission source, developed in the field of geophysics.

neural networks—algorithms loosely modeled after the human brain designed to detect patterns in data.

nondestructive testing—operator-controlled process of measuring structural and material properties of a system that causes no structurally significant damage to the concrete.

pattern recognition—automated recognition of patterns, see **neural networks**.

pencil lead break—standardized source to produce an acoustic emission having a quasi-step function.

piezoresistivity—change of the electrical resistivity of a material in response to applied mechanical strain.

potentiometer—displacement sensor based on the principal of voltage division.

radar—radio detection and ranging; a technique that employs transmission of radio waves of known frequency and time of transmission and detection of their reflections to detect and infer movement of remote objects over time.

scatterer—physical object from which radar or acoustic energy is reflected or refracted, typically at boundaries between media having different dielectric properties or acoustic impedance, respectively.

sensor—device, subset of transducer having the ability to sense (but not transmit).

structural health monitoring (SHM)—methodology entailing the use of instruments and sensors to identify changes of structural and material responses, environmental conditions, and loads.

transducer—device that can both sense and transmit a physical process.

vibrations (structural)—dynamic motion of a structure caused by live loading, wind, seismic activity, or various construction tasks; differentiated from vibrations used to consolidate fresh concrete.

visual inspection—examination of the visible surfaces of a structure with the objective to recognize and classify different types of damage and identify the probable cause of the observed distress.

weigh-in-motion (WIM)—process that uses devices to capture and record the axle and gross vehicle weights when vehicles drive over them at highway speed.

CHAPTER 3—CONSIDERATIONS FOR IMPLEMENTATION OF SHM SYSTEMS

3.1—Purpose of structural health monitoring (SHM)

3.1.1 Introduction—Structural health monitoring (SHM) entails the use of instruments and sensors to gather structural performance data from a structure over a period of time. The principal purpose of SHM is to monitor changes in the structure's response, which could be related to the occurrence or progression of damage, and, ultimately, to establish a measure of structural condition or health. The resulting information is used by owners and asset managers to make decisions regarding maintenance and repair of the structure. In this report, a wide range of approaches and technologies are considered as part of SHM, such as:

(a) Short-term monitoring (days to weeks) during load testing, where measurements are usually repeated every year or so for comparison

(b) Long-term monitoring (months to years) to capture response changes caused by aging and deterioration

(c) Monitoring with fixed installed sensors or sensors with the options of both wired and wireless technologies.

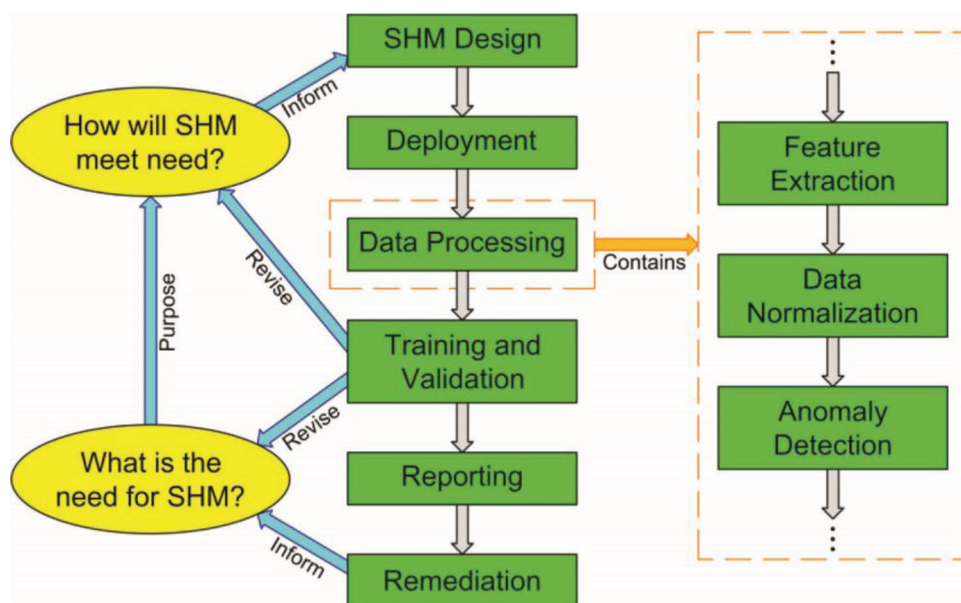


Fig. 3.1.2—Illustration of structural health monitoring process.

- (d) Contactless monitoring using remote sensing technologies
- (e) Global monitoring to capture changes in modal properties—that is, natural vibration frequencies and mode shapes
- (f) Local monitoring to capture changes in material properties caused by fracture, corrosion, or material degradation processes

In this document, SHM is defined to be distinct from nondestructive testing (NDT) primarily in terms of scope and convention. NDT refers to periodic or specially scheduled nondestructive/non-invasive measurements to examine a specific characteristic of a structure that is normally confined to a specific local area. SHM more broadly employs data collection over time, not necessarily on-demand but possibly continuous or real-time, to establish the structural condition over time. Another distinction is that SHM is typically passive—that is, the response due to often unknown or uncontrolled stimuli is monitored—whereas NDT is usually active—that is, the response caused by a known stimulus is measured and analyzed. However, these definitions are not strict and universally observed, so there is some uncertainty and disagreement about them. For example, unaided visual inspection is often considered to be NDT, while contactless/vision-sensing applications, such as digital image correlation, are often classified as SHM. In practice, SHM may augment information determined by routine inspections and NDT or may inform the owner when some specific on-demand method should be employed. A recent application of SHM is their use for digital twins where data collected from in-service structures are used to update the properties of their numerical counterparts, which typically is a finite element (FE) model. Digital twins enable:

- (a) Operational assessment (for example, routine updating for material property changes due to cyclic loading)
- (b) Post-event assessment (for example, after a major load event has occurred)

- (c) Prediction of damage or failure (for example, when threshold values for certain structural properties are reached)

Regardless of the definitions above, this document does not discuss technologies already addressed by ACI Committee 228 and its documents (ACI 228.1R, ACI 228.2R).

3.1.2 Basic considerations to implement a structural health monitoring system—Implementation of an SHM system requires the collaboration of multiple stakeholders (for example, owner, engineer, contractor, data analyst) to meet a need for structural maintenance and safety. SHM systems are engineered systems designed according to a process, an illustration of which is shown in Fig. 3.1.2, starting with identification of the goals of the system and ending in stakeholder action. While the focus of this report is specifically on sensor technologies for SHM design, the process is briefly discussed in this section, as the monitoring process may inform which sensors are selected.

Prior to its design, the need for the SHM system must be established (“What is the need for SHM?”). Objectives of the SHM system may include: 1) validate or assure that the structural system behaves according to design assumptions; 2) track the change in condition or response of a known critical structural element (for example, a bridge beam); or 3) inform scheduling for maintenance and remediation actions throughout the service life of the structure. The specific objective will often be determined by the owner’s needs and expectations for a return on investment. A set of measured structural responses are then proposed to meet the specific objective (“How will SHM meet need?”). SHM design then considers the following components:

- (a) Physical infrastructure, such as sensor types and positions to best measure desired response, but also including computing and data management resources
- (b) Plan for installation and deployment, which may depend on accessibility of the structure, availability of

Table 3.1.3—Sensor selection table with references to corresponding sections in Chapters 4 and 5

Measurement entity (in alphabetical order)	Alternative 1		Alternative 2		Alternative 3	
Acceleration	Accelerometer (structural vibration)	4.10	Acoustic emission sensor (stress wave propagation, acceleration-sensitive sensor)	4.1		
Acoustic emission	Acoustic emission sensor	4.1	Accelerometer	4.10		
Chloride content	Chloride sensor	5.1				
Crack motion	Potentiometer	4.4	Fiber optic sensor	4.5		
Cracking/fracture	Acoustic emission sensor	4.1	Conductive surface sensor	4.2	Fiber optic sensor	4.5
Corrosion	Corrosion sensor	5.2				
Displacement (contact)	LVDT	4.4	Potentiometer	4.4	Acoustic emission sensor (stress wave propagation, displacement-sensitive sensor)	4.1
Displacement (contactless)	RADAR	4.7	LIDAR	4.6	Digital image correlation	4.3
Force/load	Load cell	4.2.4	Weigh-in-motion sensor	4.2.8		
Humidity	Humidity sensor	5.3, 5.9				
pH value	pH sensor	5.5				
Precipitation	Rain sensor	5.9				
Pressure	Piezometer (water pressure)	5.6	Barometer (air pressure)	5.9		
Solar radiation	Pyranometer	5.9				
Strain (pointwise)	Strain gauge	4.8	Fiber optic sensor	4.5		
Strain (full field)	Digital image correlation	4.3	Conductive surface sensor	4.2		
Temperature	Temperature sensor	5.7				
Tilt/rotation	Tilt sensor	4.9	Digital image correlation	4.3		
Velocity	Laser Doppler vibrometer (structural vibration)	4.10	Geophone (structural vibration)	4.10		
Wind speed	Anemometer	5.9				

support utilities such as power and communications, and desire to periodically replace or update instrumentation

(c) Data processing plan, which requires extraction of the features of interest from the raw data, normalization with respect to known inputs or non-damage phenomena, and determination whether this measured response qualifies as normal or anomalous

(d) Plan for system validation and training to ensure that the data provide reliable, accurate, and necessary information for decision-making

(e) Reporting requirements, such as what, and how often (for example, alerts versus routine reporting), information is transmitted and to which stakeholders

(f) Plan of action

All these components are considered with respect to the expected structural service life and the expected lifetime of the monitoring system, both of which inform a long-term plan regarding how the above components may change with time.

Once an SHM system is deployed, validation and training provide the feedback used to assess and revise it. Validation refers to the assessment of whether the SHM system is achieving its intended objective. An invalid system may arise if insufficient or inappropriate instrumentation is used

to monitor the behaviors of interest, if other unforeseen and unmeasured behaviors are found to better meet the monitoring objective, or if the objectives for SHM evolve over the service life of the structure. Training is used in the machine-learning sense—that is, adaptation of the monitoring system based on observed performance. Such adaptation is often necessary, for example, to establish thresholds for anomaly detection in variable environmental conditions, changing load demands on the structure, or changing structural condition.

3.1.3 Selecting structural health monitoring technologies—Selecting appropriate sensing technologies and instrumentation is critical for the successful implementation of an SHM system. Some general considerations apply to sensor selection in all cases, regardless of what is being measured:

(a) How must the sensor be installed? Must it be embedded into the concrete, tied to a reinforcing bar, or externally mounted?

(b) How many sensors are required to measure the behavior of interest?

(c) What range of response is the sensor capable of measuring? Does this encompass the anticipated range of structural behavior?

- (d) What effects, if any, do temperature and humidity have on the sensor response?
- (e) What other equipment is needed to measure the sensor response?
- (f) How is the sensor and associated equipment powered?
- (g) What are the accuracy and precision of the sensor?
- (h) At what rate can the sensor measure the response? Is measurement desired every hour, minute, second, or less?
- (i) Is the sensor robust enough to withstand the weather or being cast in concrete?
- (j) What is the expected lifetime of the sensor? Is replacement expected, or is the sensor sacrificial?

It is important to note that no sensor technology directly detects damage or condition in a structure; most sensors measure a physical quantity (for example, voltage), which may be converted to an indirect measure of structural response and further interpreted as a state of the structural damage or condition (Worden et al. 2007). It should further be noted that currently no universally agreed-upon definitions for terms such as “damage,” “condition,” or “health” exist; these are typically defined on a project basis among the stakeholders of an SHM project. Therefore, sensor selection is largely constrained by the type of desired structural response. Table 3.1.3 summarizes the sensor technologies discussed in the report with respect to the entities that may be measured. Multiple alternatives are not meant to suggest a hierarchy of the listed sensors; this decision should be made on an individual basis considering the specific strengths, weaknesses, and measurement principles of each sensor, all of which can be found in the listed sections.

Chapter 6 discusses enabling system technologies, including energy harvesting technologies (Section 6.1), sensor platforms such as those based on micro-electrical mechanical systems (MEMS) (Section 6.2), and wireless sensor networks (WSNs) (Section 6.3).

3.2—Pertinent characteristics of concrete structures

3.2.1 Introduction—This chapter introduces those properties and response characteristics of concrete structures that must be known and accounted for when selecting sensor types, positioning sensors, and interpreting SHM data. As can be seen in Fig. 3.2.1, undamaged concrete is a macroscale composite material, normally composed of mineral aggregates of different size (light gray) bound by a cement matrix (gray) that incorporates both an interconnected pore structure and discrete (unconnected) entrapped and entrained air voids (black), which contains either pore fluid or air. Concrete is a quasi-brittle material and, as such, is relatively strong under compressive stresses but weak under tensile stresses. To overcome this weakness, designers add reinforcement in the tension zones of a structural member, typically in the form of steel reinforcing bars. Concrete and steel are compatible in that they exhibit similar thermal expansion characteristics, and the chemical nature of concrete protects the steel from atmospheric corrosion. Their responses to applied loads and other environmental conditions are, however, significantly different, which directly affect the response of structural

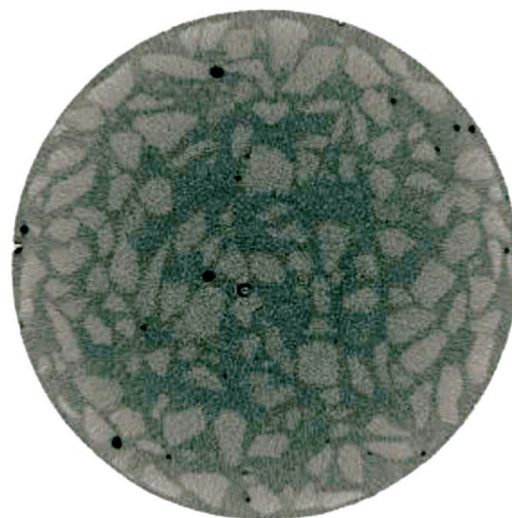


Fig. 3.2.1—Internal structure of undamaged concrete visualized using computed tomography (CT) scan (photo courtesy of University of Burgos, Spain). (Note: The diameter of the section is 100 mm [3.94 in.].)

concrete members. Unreinforced concrete structures are not discussed in this chapter.

3.2.2 Material characteristics—The constituents that comprise structural reinforced concrete are discussed in this section.

3.2.2.1 Concrete—This section discusses hydraulic cement concretes, which are primarily based on portland cement binders and a range of additives that modify the fresh and hardened properties of the material. The most important characteristics of concrete that have a direct influence on the SHM of concrete structures are the following:

(a) Early-age variations during the hydration process: Sensors embedded inside the concrete (either in direct contact with concrete or attached to a reinforcing bar) should be designed to withstand thermal variations and a basic chemical environment without damage.

Once the different components of concrete are mixed, compounds within the portland cement known as calcium silicates react with water to form calcium-silicate-hydrates (CSHs). The hydration reaction progressively provides stiffness and strength to the concrete, changing it from a malleable to solid material within hours. This causes a strong modification of the mechanical parameters over time (Bernard et al. 2003; Boulay et al. 2014; Granja et al. 2014; Azenha et al. 2010; Maia et al. 2011).

Hydration reactions are exothermic and result in significant heat evolution within the concrete at early age (Fig. 3.2.2.1a). Depending on the cement and admixture chemistry, boundary conditions, and concrete volume and geometry, an internal temperature up to 80°C (175°F) can be reached. The chemical reactions also create pore fluid within concrete that is highly basic (high pH) in nature.

The evolution of the hydration reaction changes during the first hours and days after mixing, where high heat evolution generally correlates with strength and stiffness development. The concrete remains in the malleable, fresh state

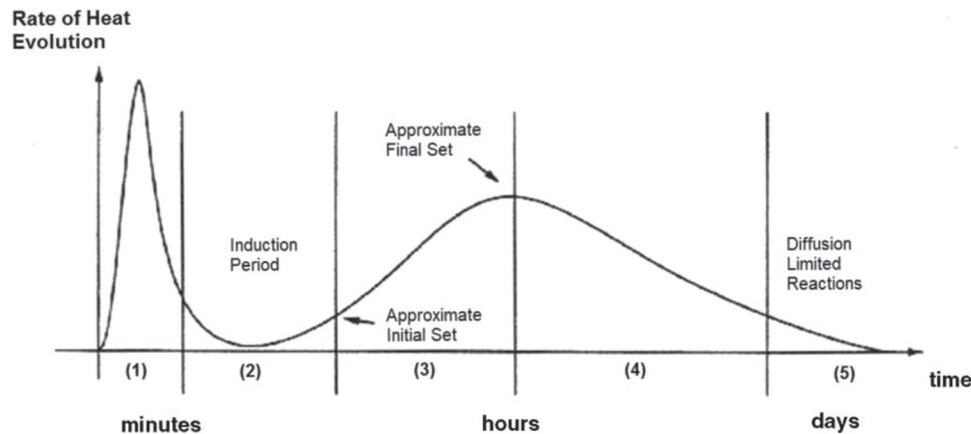


Fig. 3.2.2.1a—Heat evolution of concrete during the hydration of portland cement with different stages of the hydration reaction indicated by numerals (Nelson 1990).

through hydration Stages 1 and 2, and starts to stiffen and set at the beginning of Stage 3. During Stage 3, the compressive strength of concrete is low and its tensile strength is almost nonexistent. Consequently, surface cracking may occur that is induced by thermal or moisture gradients within the material (ACI 213R; Branco et al. 1992). Concrete continues to gain strength in Stages 4 and 5, but at a decreasing rate.

(b) Intrinsic heterogeneous mesostructure: Typically, theoretical analyses of concrete behavior are carried out assuming that concrete is a homogeneous material, and this is a reasonable assumption at the macroscale level where average values are used. For situations in which it is necessary to measure local response entities, such as strain, moisture, and pH, at a point, the heterogeneous nature of concrete cannot be ignored and may dictate sensor selection, data analysis methods, and data interpretation. At this local scale, the exact position of a sensor may have a relevant influence on the measured values, especially when the sensor exhibits a size similar to the aggregates.

Concrete is a composite material composed of coarse aggregates (with dimensions on the order of tens of millimeters) bound by a matrix composed of fine aggregates, cement, and water. The hydration process that gives rise to a solid material is described in Subsection (a). The mechanical properties of hardened concrete, from a macroscopic point of view, are a combination of the mechanical properties of the different components (Neville 1996) that directly affect the stress-strain relationship. In fact, the mechanical response (for example, stress-strain relationship of the concrete as a composite response) can be thought of as an intermediate between those of the coarse aggregate skeleton and that of the hardened cement paste (refer to Fig. 3.2.2.1b). However, the influence of voids and the interfacial transition zone between the paste matrix and individual aggregate particles, as well as other phenomena such as microcracks, cause the relationship to be more complex than a simple weighted average of constituent properties, as demonstrated by the nonlinearity of the concrete stress-strain curve.

(c) Modulus of elasticity and compressive strength: Deterioration processes, which typically result in a decrease

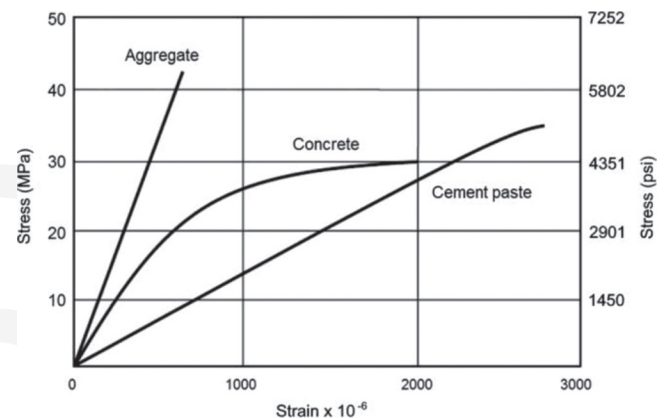


Fig. 3.2.2.1b—Stress-strain relations for cement paste, aggregate, and concrete (adapted from Neville [1996]).

in the macrolevel stiffness, may act to counteract the inherent increase in stiffness over time.

The modulus of elasticity (E_c) and compressive strength (f'_c) of concrete continually increase over time as a function of the hydration process, although the rate at which they change may not be the same (ACI 209.2R). ACI 318 suggests the following relationship between E_c and specified 28-day compressive strength in response to applied mechanical strength f'_c for normalweight concrete

$$E_c = 4700\sqrt{f'_c}, \text{ where } E_c \text{ and } f'_c \text{ are in MPa} \quad (3.2.2.1)$$

$$E_c = 57,000\sqrt{f'_c}, \text{ where } E_c \text{ and } f'_c \text{ are in psi}$$

The time-dependency of f'_c impacts long-term monitoring applications where measurements of strain are to be related to stress or deformation and computed by using the modulus of elasticity (E_c).

(d) Nonlinear stress-strain behavior: The nonlinear stress-strain behavior of concrete is important from an SHM standpoint. For example, measurements of strain in concrete are usually used to calculate stress. In this case, the use of a stress-strain relationship is required. If it is necessary to correlate measured parameters of a concrete structure, such as vertical deflection, rotation, and natural frequency, with

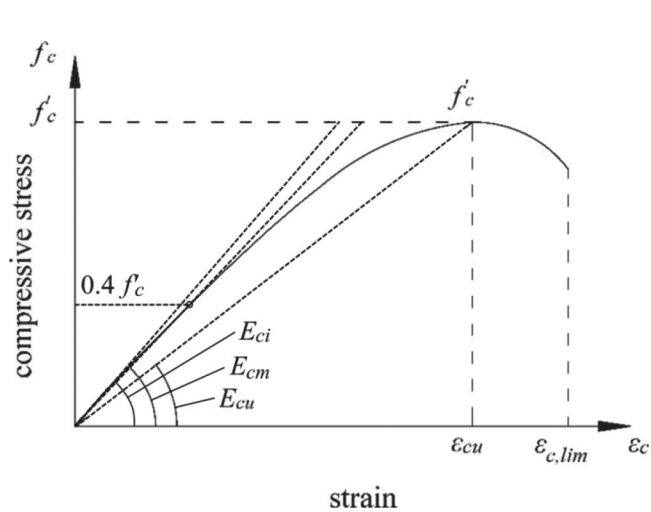


Fig. 3.2.2.1c—Schematic stress-strain relationship for quasi-static loading in uniaxial compression (adapted from fib Model Code 2010).

theoretical ones—for example, those obtained from finite element (FE) models—a well-defined stress-strain relationship is necessary.

Concrete shows significantly different mechanical response in compression compared to tension—that is, the tensile strength of concrete is typically on the order of 10 percent (or even less) of its uniaxial compressive strength. Thus, for strength calculations of reinforced concrete, the tensile strength of concrete is typically neglected. As will be discussed in Section 3.2.3, tensile forces are supported by the steel reinforcement in a reinforced concrete element. From a structural point of view, the behavior of concrete under compressive stress is more important. Recall that the stress-strain relationship of concrete under compression is nonlinear. fib *Bulletin 42* (fib 2008) provides theoretical parabolic stress-strain relationships for concrete in compression, which are valid for regular as well as high-performance concrete. Figure 3.2.2.1c illustrates a schematic stress-strain curve of concrete in compression.

ACI 318 does not define a theoretical stress-strain relationship; it only considers that, under service loads, the behavior of concrete is linear-elastic (and its modulus of elasticity is defined) and, under failure loads, a rectangular compression block may be considered in place of the more realistic parabolic curve shown in Fig. 3.2.2.1c. In the fib Model Code 2010, two different definitions for the modulus of elasticity (E_c) are given: the instantaneous modulus of elasticity (E_{ci}), defined as the tangent of the stress-strain curve at its origin; and the secant modulus of elasticity (E_{cu}), defined as the slope of the line connecting the origin with the peak of maximum compression (f'_c). Eurocode 2 defines a secant modulus of elasticity inside the elastic range of the concrete (where linear-elastic behavior is almost true) (E_{cm}), defined as the slope of the line connecting the origin with the point of the curve corresponding to a stress value of $0.4f'_c$ (shown in Fig. 3.2.2.1c). Both the fib Model Code 2010 as well as Eurocode 2 suggest that the following can be assumed: $E_{ci} = E_{cm}$. ASTM C469/C469M testing provides the modulus of elasticity from a compression test on concrete cylinders.

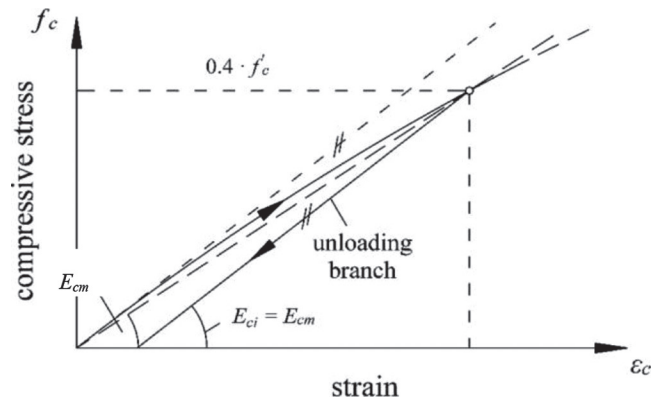


Fig. 3.2.2.1d—Illustration of stress-strain diagram for concrete showing loading and unloading branches (Model Code 2010).

The slope of the stress-strain curve is also different for loading and unloading. For the unloading branch, linear-elastic behavior of concrete is assumed, and the slope of this curve is E_{ci} (Fig. 3.2.2.1d). As can be observed, unloading from some stress level to zero stress leads to residual strain, which is largely attributed to microcracking. This residual strain is larger when unloading from a higher stress. The effect of cyclic loading is discussed in more detail in Subsection (f).

Finally, the stress-strain response of concrete is also strain-rate-dependent (Zhou and Chen 2013; Cadoni et al. 2009; Bischoff and Perry 1991, 1995; Fu et al. 1991; Dilger et al. 1984). Most of the international standards do not discuss this topic thoroughly. A relevant exception is Model Code 2010, which shows formulas to modify, among others, the modulus of elasticity as a function of the loading rate.

(e) Time-dependent volume changes—creep and shrinkage: Time-dependent volume changes have a strong influence on long-term SHM because they occur simultaneously with all other deformations. It is important to recognize that creep and shrinkage cause internal forces only in structures where deformations are restrained. Separating the deformations due to external loads and creep and shrinkage is difficult because there are no concrete stress-strain relationships that can reliably predict the complex nonlinear visco-elasto-plastic behavior of concrete. Not considering these time-dependent volume changes may lead to incorrect estimation of the stresses in the structure.

Concrete exhibits time-dependent volume changes caused by creep and shrinkage. Both phenomena have received significant attention since the 1970s. Figure 3.2.2.1e illustrates a simple example of the strain changes caused by creep and shrinkage with a sudden load applied at time x .

Today's consensus on the physical phenomenon behind shrinkage is that it is a result of progressive loss of water from the cement paste caused by evaporation and hydration of the cement. However, there is no consensus regarding causes of creep. This is evident in ACI 209.2R, where four different models are proposed to calculate creep in concrete. Creep has received significant attention over the last 50 years (Zienkiewicz and Watson 1966; Madsen and Pžent 1983; Bažant and Panula 1980; Neville et al. 1983; Vandewalle

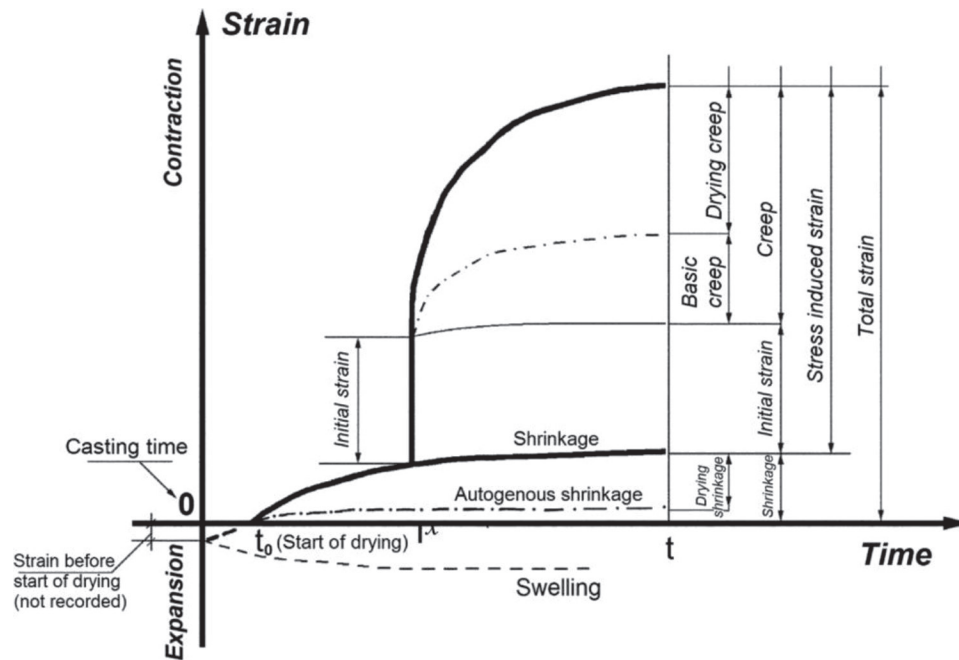


Fig. 3.2.2.1e—Variation of strain with time due to creep and shrinkage, considering an applied compression load at time x (ACI 209.1R).

2000; Bažant et al. 1995; Bažant 2001; Mazloom 2008). It is accepted by most experts that creep has two components: basic creep and drying creep. Basic creep is defined as the creep resulting from a concrete element under sustained load and constant internal moisture content. Drying creep is defined as the additional creep caused by loss of moisture. Creep and shrinkage result in progressive increases of deflections and rotations in concrete structures. Additionally, in post-tensioned concrete structures, time-dependent effects result in gradual loss of the prestressing force. In addition, internal force redistribution occurs for the case when concrete structures are built with varying concrete qualities and/or ages.

(f) Effect of cyclic loading on modulus of elasticity: The concrete's fatigue response has a significant impact when SHM is applied to measure cyclic loading. For real structures subjected to cyclic loading, the combined effect of an increase of the modulus of elasticity with age and a decrease of the modulus of elasticity with cyclic loading and initiation of degradation mechanisms occurs. In this case, the exact quantification of the modulus of elasticity at each instant is difficult. Nevertheless, it is necessary to be aware that both phenomena occur simultaneously. Depending on the variation of concrete hardening on the one hand and the degradation processes due to cyclic loading on the other hand, the final result can be a global increase of the modulus of elasticity, a global decrease of it, or an intermediate result.

Cyclic loading induces incremental distributed damage throughout the concrete because of a progressive growth of microcracking and, as a consequence, a progressive loss of stiffness and strength. Many international standards and guides (for example, ACI 215R, Eurocode 2 2004, and Model Code 2010) consider the effects of cyclic loading in terms of the number of loading cycles that cause failure

when a certain load level is applied—for example, through the use of S-N (that is, difference between maximum and minimum stress versus number of stress cycles) curves. The evolution of the mechanical parameters of concrete (compressive strength and modulus of elasticity, among others) as a function of cyclic loading has been studied both directly and indirectly (Hsu 1981; Do et al. 1993; Kim and Kim 1996; Zanuy et al. 2009; Vicente et al. 2014). The research has shown that cyclic loading causes a progressive reduction of the modulus of elasticity. The evolution of the modulus of elasticity up to failure can be divided into three regions, as shown in Fig. 3.2.2.1f. The first region, from the origin to a cycle ratio equal to 0.1, shows a rapid decrease (between 10 and 30 percent, depending on the maximum stress). The second region, from a cycle ratio of 0.1 to 0.8, shows a more gradual linear decrease, and its slope depends on maximum cyclic stress. Finally, the third region, from 0.8 up to failure (cycle ratio equal to 1.0), shows again a rapid decrease to failure.

In Fig. 3.2.2.1f, N is the number of applied cycles, N_f is the total number of cycles to failure (also called fatigue life), S_{max} is the maximum stress ratio (that is, the ratio of maximum stress applied to the strength), and S_{min} is the minimum stress ratio (that is, the ratio of minimum stress applied to the strength). The fatigue life strongly depends on S_{max} and S_{min} as well as on the frequency of applied load cycles. It should be noted that fatigue life data show great scatter. Also, the curves shown in Fig. 3.2.2.1f are idealized curves for a consistent level of loading over the range of cycles. In real (monitored) structures, the loading magnitudes and frequencies are more random.

3.2.3 Reinforcement—Typical reinforcements used in concrete structures include the following.

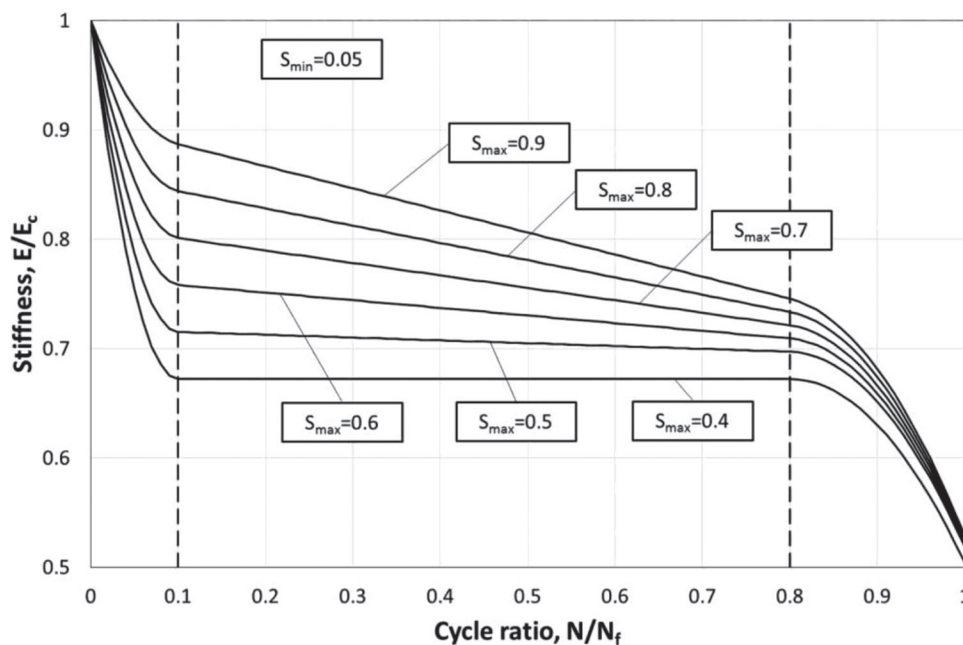


Fig. 3.2.2.1f—Relative modulus of elasticity as a function of applied cyclic stress (Zanuy et al. 2009).

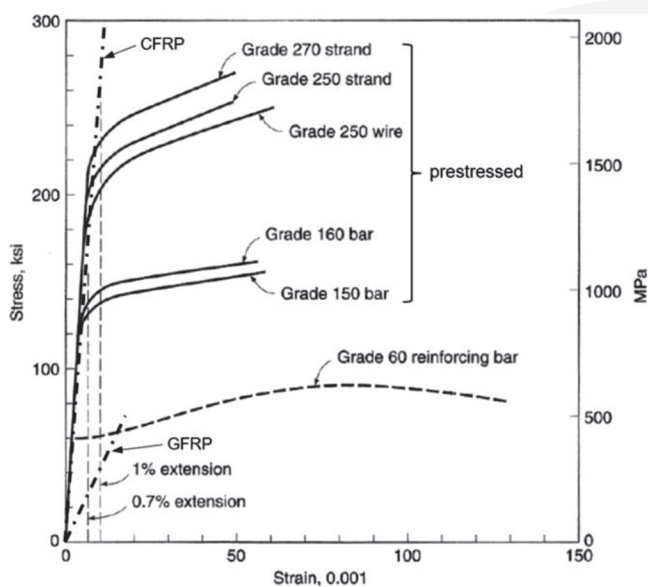


Fig. 3.2.3—Stress-strain curves for typical steel and polymer fiber composite (FRP) reinforcements (adapted from Fig. 2.16 of Nilson [2010]).

- (a) Carbon steel reinforcing bars (ASTM A615/A615M, ASTM A706/A706M, ASTM A1035/A1035M)
- (b) Epoxy-coated steel reinforcing bars (ASTM A775/A775M, ASTM A934/A934M)
- (c) Galvanized steel reinforcing bars (ASTM A767/A767M)
- (d) Stainless steel reinforcing bars (ASTM A955/A955M)
- (e) Structural steel sections (ASTM A36/A36M, ASTM A242/A242M, ASTM A572/A572M, ASTM A588/A588M, ASTM A992/A992M)
- (f) Prestressing strands or wires (ASTM A416/A416M, ASTM A421/A421M, ASTM A722/A722M)

(g) Fiber-reinforced polymer (FRP) reinforcing bars (ASTM D7205/D7205M)

The stress-strain curves for reinforcing steel are well defined and typical results are shown in Fig. 3.2.3. The modulus of elasticity can be assumed as $E_s = 29,000$ ksi (200,000 MPa) for steel bars—that is, reinforcements (a) through (e). For seven-wire strands used in prestressed concrete applications, a modulus of elasticity is in the range of $28,500$ ksi (196,500 MPa) $\leq E_p \leq E_s$ is typically assumed (where E_p is the modulus of elasticity of steel strands and wires used for prestressed and post-tensioned concrete elements, and E_s is the modulus of elasticity of steel bars). The modulus of elasticity for FRP reinforcements vary greatly, depending on the type of fiber used. Generally, carbon fiber-reinforced polymers (CFRPs) are significantly stiffer than glass FRPs (GFRPs), with the former also having much higher strengths. The main differences between steel and FRP reinforcements is that the former can plastically deform significantly because it yields under increasing load. The failure strains are therefore much larger—up to 13 to 15 percent for Grade 60 ($f_y = 420$ MPa [60 ksi]) steel reinforcing bars.

3.2.4 Member structural response—For SHM, the following points should be considered:

- (a) Cracking introduces significant strain and curvature variations in a member and significantly affects the vibration and mechanical wave propagation response.
- (b) Sensors such as strain or displacement gauges should not be placed within the development length or a splice (unless studying the behavior of these regions is of particular interest) because the data obtained are not representative of the structure but of this particular region.
- (c) For strain measurements performed on the reinforcement, measured values are strongly influenced by their location in relation to cracks in concrete. If the reinforcement is encapsulated in the concrete, strain sensors

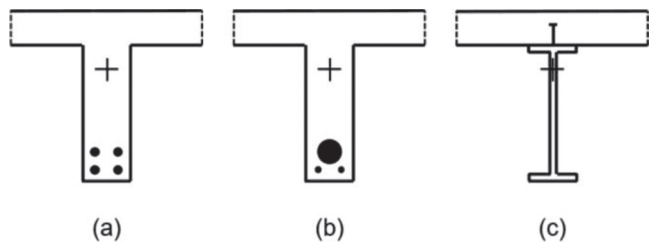


Fig. 3.2.4a—Typical schematic cross sections of beam member: (a) nonprestressed reinforced (steel or FRP bars); (b) prestressed; and (c) steel-concrete composite. Small circles indicate bars and large circles tendons. (Note: Only the required tension reinforcement for a section experiencing a positive bending moment (tension at the bottom) is shown for simplicity.)

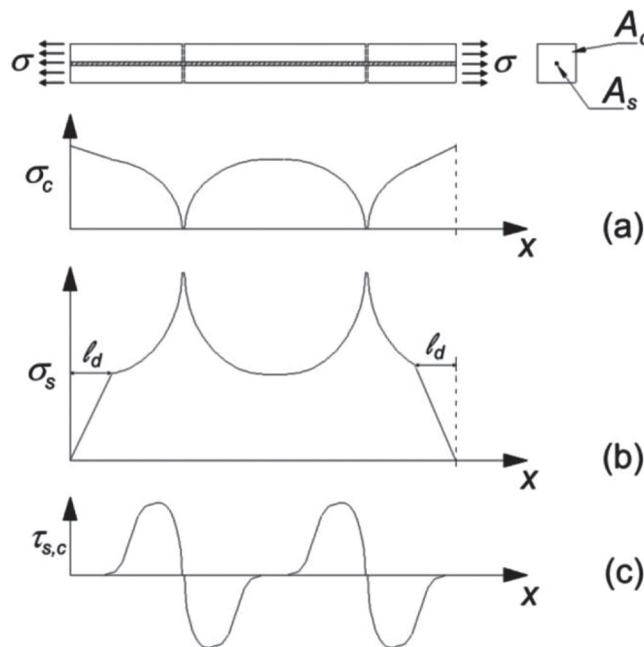


Fig. 3.2.4b—Illustration of axial stress in: (a) concrete and (b) reinforcement; and (c) shear stress at the interface for the case of a tension member loaded in uniaxial tension.

on reinforcement must be placed prior to casting. As the exact locations of future cracks cannot be known, the results obtained by such sensors have a high degree of uncertainty. An option, however, is to mount surface sensors such as strain gauges to the reinforcement steel after cracking has occurred, preferably at the crack locations. Local damage to the concrete cover requires careful patching to prevent subsequent reinforcing bar corrosion.

The structural response of a reinforced concrete member depends significantly on what type of reinforcement is used and whether the member is prestressed or not. Figure 3.2.4a illustrates some typical cross sections for beam members. For the case of nonprestressed members (Fig. 3.2.4a(a) and (c)), cracking in the concrete is usually observed under service-level conditions and indicates that the reinforcement has been activated. For the case of prestressed members (Fig. 3.2.4a(b)), cracking may be a sign of significant over-

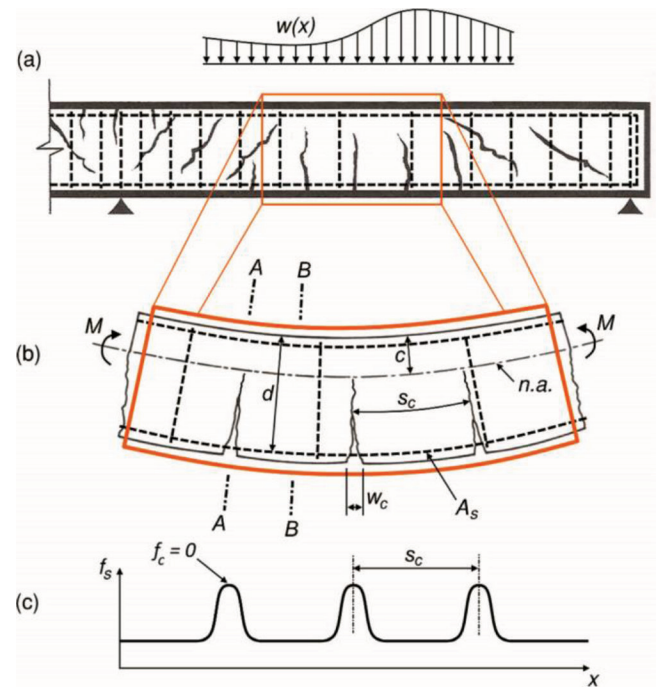


Fig. 3.2.4c—Effect of cracking on load-sharing between concrete and reinforcement: (a) continuous reinforced concrete beam with typical crack pattern and reinforcement (dashed lines) (adapted from ACI 318-14); (b) deflected shape of member portion under bending; and (c) variation of axial stress in tension reinforcement along member length. (Notes: c is depth of neutral axis; d is distance between extreme compression fiber and centroid of flexural tension reinforcement; A_s is area of flexural tension reinforcement; s_c is estimated crack spacing; w_c is estimated crack width; f_s is reinforcing bar stress; and f_c is concrete stress.)

load, loss of prestress, or steel section caused following corrosion.

Cracks in concrete result in a substantial modification of the strain distribution and, consequently, the stress distribution. This modification is especially intense near the crack, in both concrete and steel reinforcing bars. Figure 3.2.4b illustrates load sharing between concrete and reinforcement along the length of a tension member with cracked concrete. It should be noted that it is impossible to know, a priori, the exact position, path, length, and width of each crack, because they are strongly influenced by the concrete's heterogeneity. Also indicated in Fig. 3.2.4b(b) is the development length ℓ_d , which is the embedment length required to develop the design strength of the bar. In other words, this is the length required to transfer the applied stress from the concrete into the reinforcement, which occurs via shear stresses at the interface between steel and concrete (Fig. 3.2.4b(c)).

Figure 3.2.4c visualizes the implications of cracks on the strain, and hence stress, variations in the reinforcement for a flexural member. Although this figure only shows the effect on stress variations in the flexural tension reinforcement, the effect is similar for shear reinforcement at diagonal cracks.

At the macroscopic scale, cracking causes a reduction in stiffness of a structural member, that is, the curvature of

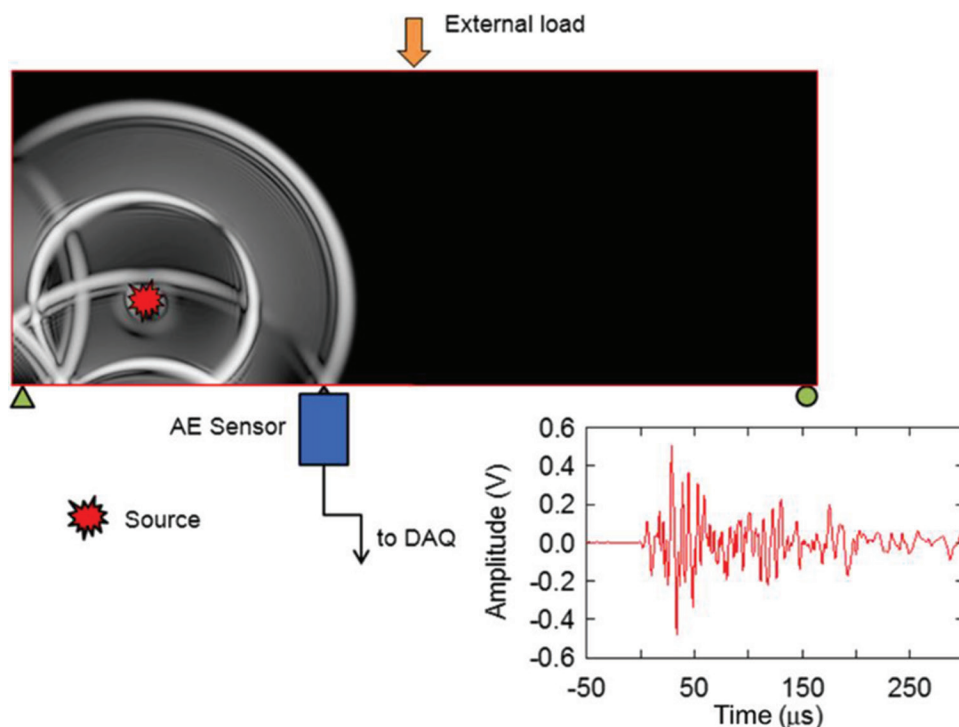


Fig. 4.1.1—(a) Principle of acoustic emission; and (b) example of typical recorded signal. The snapshot of the propagating stress wave was created using Wave2000 (Cyberlogic 2013). (Note: DAQ stands for data acquisition system.)

a section between cracks (“B” in Fig. 3.2.4c(b)) is significantly less compared to the curvature at a crack location (“A” in Fig. 3.2.4c(b)). This has consequences for several parameters, such as vertical deflections, rotations, and natural vibration frequencies.

CHAPTER 4—SENSORS FOR STRUCTURAL RESPONSE

This chapter describes sensor technologies used to measure the response of a structural system, including:

(a) Acoustic emission sensors (Section 4.1) measure transient stress waves in concrete. These sensors are suited for detecting micro- and macro-level cracking, concrete crushing, and reinforcement yielding and fracture.

(b) Conductive surface sensors (Section 4.2) are electrically conductive materials applied to the surface of a concrete member. They are used to detect strain and cracking at the surface, but not the interior, of a member.

(c) Digital image correlation (Section 4.3) is a visual (that is, non-contact) method for measuring displacements in two or three dimensions. This technology is capable of full-field measurement, and the displacements may be translated to strains using post-processing.

(d) Displacement sensors (Section 4.4) presents two common types of contact sensors for measuring displacements: potentiometers and linear variable differential transformers (LVDTs).

(e) Fiber-optic sensors (Section 4.5) may be used to measure strain in two unique ways: long-gauge and distributed sensing. While this section focuses exclusively on strain measurements, fiber-optic sensors may also be used

to measure a variety of phenomena, including displacement, inclination, acceleration, humidity, pH, and temperature.

(f) Light detection and ranging (LIDAR) (Section 4.6) is a remote sensing technique that measures distances to objects. Often used for surveying, terrestrial LIDAR systems may also be used to measure displacement.

(g) Synthetic aperture radar imaging (Section 4.7) is another remote sensing technique to measure distances and changes in position.

(h) Strain sensors (Section 4.8) present two common types of contact sensors for measuring strains: resistive strain gauges and vibrating-wire (VW) strain gauges.

(i) Tilt sensors (Section 4.9) measure angles with respect to either the vertical or horizontal plane. This may be done using vibrating-wire, electrolytic, MEMS, or pendulum-based sensors.

(j) Vibration sensors (Section 4.10) present sensors for measuring acceleration and velocity that characterize dynamic motion. This includes laser Doppler vibrometers, accelerometers, and geophones.

The entities these sensor technologies measure are also summarized in Table 3.1.3.

4.1—Acoustic emission (AE) sensors

4.1.1 Physical basis and property inferred—Acoustic emissions (AEs) are the result of sudden releases of strain from fracture processes within a solid body (Grosse and Ohtsu 2008). A stress wave initiated at the source origin travels through the solid medium to the surface of the body where it can be recorded by sensors, as illustrated in Fig. 4.1.1. This process is irreversible and therefore not repeatable. Other names for AE include stress wave emis-

Table 4.1.1—Expected sources of AE in structural concrete

Phenomenon	Cause(s)	Type*
Microlevel cracking (distributed)	Shrinkage, temperature, creep, low-level stresses	P
Macrolevel cracking (localized)	Tensile stress, temperature	P
Concrete crushing	Compression stress	P
Reinforcement yielding and fracture	Tensile stress, overloads, low-cycle fatigue	P
Fracture of wires in internal and external multi-wire strand systems	Corrosion, oversteering, overloading	P
Debonding and slip of reinforcement	Cyclic stresses due to live loads	S
Cracks from reinforcement corrosion	Tensile stresses due to steel expansion	S
Crack surface interaction or rubbing	Cyclic stresses due to live loads	S

*P = primary source; and S = secondary source.

sions or microseismic activity. The term “AE” is also used to define transient elastic waves generated by the release of energy within a material or by a process (EN 2000). Acoustic emission is commonly referred to as both the phenomenon as well as the monitoring technique. The physical basis is wave motion in solids (Graff 1991). The frequency content of AE signals in concrete ranges between several 10 kHz and several hundred kHz (Schumacher 2008). With one or several sensors mounted to the surface of the monitored element, the surface motions caused by these transient stress waves are detected. Typical AE sensors operate based on the principle of piezoelectricity whereby the surface motions are converted into measurable electrical signals (refer to Section 4.1.2.1 for details). To conclude, the AE technique records the emitted stress waves produced by a sudden change of the internal integrity of a structural element.

The sources of AE in concrete are mainly associated with fracture processes, such as formation and propagation of cracks. These cracks form when the tensile capacity of the concrete is exceeded. In reinforced and prestressed concrete structures, the source types presented in Table 4.1.1 can be expected (adapted from Schumacher [2008]), where P and S represent primary (new fracture) and secondary (interactions between existing concrete crack surfaces or concrete-steel interactions) sources, respectively.

Some examples of properties inferred from AE measurements are:

(a) Cracking moment indicating reduction of member stiffness

(b) Location and type of occurring concrete cracks

(c) Location of wire/strand fractures

(d) Slip or loss of bond between reinforcing and concrete

(e) Reduction of reinforcement area due to corrosion

4.1.2 Equipment needed—To perform AE measurements, the following equipment needed is described in the following sections:

(a) Sensors

(b) Preamplifiers

(c) Data acquisition hardware

(d) Computer to store the recorded signals and parameters

(e) Software to process and analyze the recorded signals and parameters

4.1.2.1 Sensors—Traditional AE sensors are piezoelectric-based devices that convert the small surface motions caused

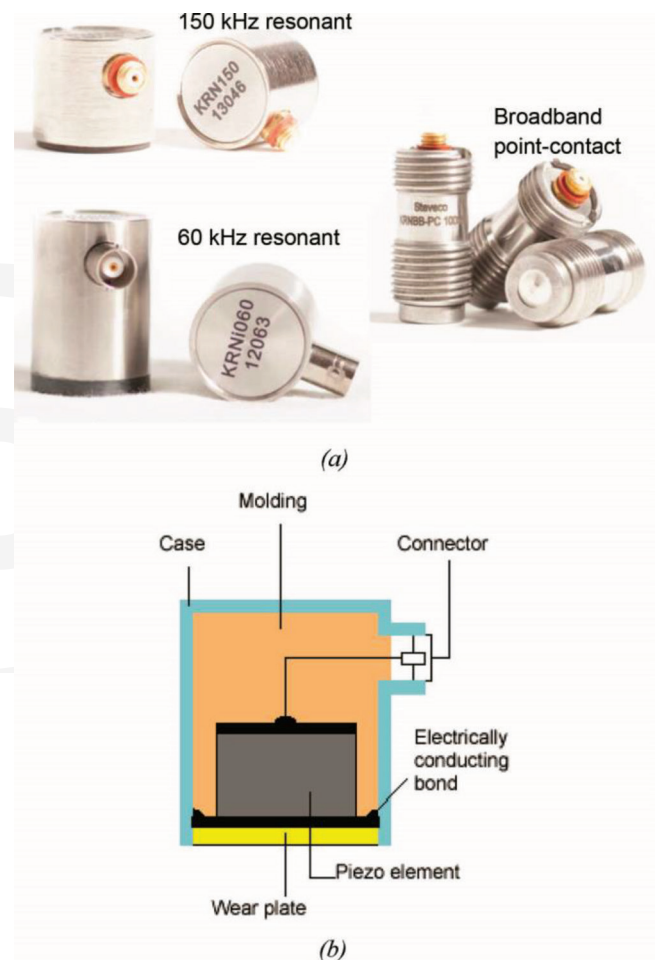


Fig. 4.1.2.1a—(a) Examples of typical AE sensors (<http://www.krnservices.com>); and (b) components of a typical sensor (Vallen 2002).

by AE into electrical signals that can be measured by a data acquisition system (DAQ). The most common material used for the piezoelectric element in the sensor are lead zirconate titanate (PZT) ceramics (Miller et al. 2005). Many different types of sensors are commercially available, depending on requirements such as type of material to be monitored, desired frequency range, and operation temperature. Examples of commercially available AE sensors and their components are illustrated in Fig. 4.1.2.1a. For applications in concrete, sensors that are sensitive in the frequency range

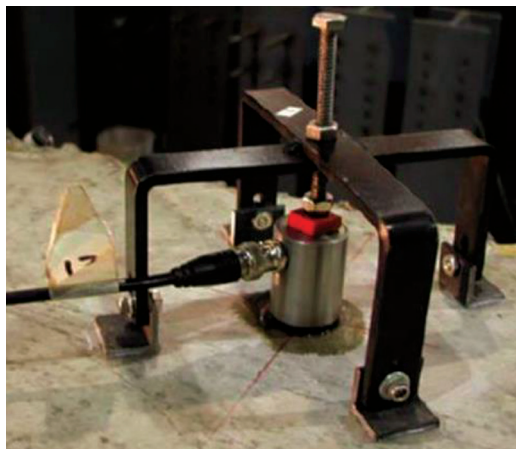


Fig. 4.1.2.1b—Example of custom-made sensor fixture for application on concrete.

between 20 and 500 kHz are typically employed (Grosse and Ohtsu 2008). Alternatively, standard ultrasonic transducers can be used. Generally, AE sensors may be categorized as resonant or broadband sensors. Resonant sensors have a strongly biased response but are sensitive and thus ideal for detecting and locating AE under real-world conditions. Broadband sensors ideally have a flat frequency response curve, producing a high-fidelity measurement. This enables them to capture the true characteristics of an AE, allowing for quantitative analyses such as moment tensor inversion (MTI) (Linzer et al. 2015). Depending on design and geometry of the piezoelectric material, the output signal is proportional to either displacement, velocity, acceleration, or a combination thereof. The response characteristics of a sensor are described by its calibration curve, which can be obtained from the sensor manufacturer. An example of an absolute sensor calibration for a range of AE sensors is provided by McLaskey and Glaser (2012).

Proper acoustic coupling of the sensor should be ensured by any of the following methods, depending on the sensor type, aperture (that is, diameter of piezoelectric sensing element), and surface conditions. Some general guidance can be found in ASTM E650/E650M. The most commonly used methods for application on concrete surfaces are described in the following:

- (a) Wet-coupling using a high-vacuum-type grease, ideal for applications on metal surfaces
- (b) Dry-coupling, for point contact-type sensors (for example, Glaser/NIST sensor [McLaskey and Glaser 2012])
- (c) Bonded using hot glue or epoxy, ideal for applications on concrete surfaces

For the first two methods, custom-made mechanical fixtures are typically used to keep the sensors in place during monitoring; an example is shown in Fig. 4.1.2.1b.

Prior to the actual test, pencil lead breaks (PLBs) can be performed in proximity to each sensor to determine adequate and consistent coupling (RILEM Technical Committee 2010). PLBs represent a repeatable high-amplitude step-type AE source that is well suited for this purpose.

4.1.2.2 Preamplifiers—Preamplifiers intensify the output signal from the sensor so that it can be measured by the DAQ. Some sensors are designed/equipped with built-in preamplifiers (Glaser et al. 1998) that allow for direct connection. Signal amplitudes are often reported in decibels (dB) based on the following relationship

$$G[\text{dB}] = A_o[\text{dB}] - A_i[\text{dB}] = 20 \log_{10} \left(\frac{A_o[\text{mV}]}{A_i[\text{mV}]} \right) \quad (4.1.2.2)$$

where G is the amplification factor (or gain), and A_i and A_o are sensor input and output voltages, respectively.

4.1.2.3 Data acquisition hardware—Acoustic emission (AE) data acquisition systems (DAQs) mainly consist of an analog-digital converter (ADC) to digitize the electrical signals produced by the sensors. Numerous specialized AE DAQs are commercially available and commonly have the following capabilities:

- (a) 12- to 18-bit ADC
- (b) Sampling rates up to 40 MHz
- (c) Triggered recording based on predefined criteria such as signal voltage threshold exceedance, signal onset slope, and minimal signal duration
- (d) Event-based recording to allow individual signals to be grouped to events (some systems do this during post-processing)
- (e) Automatic extraction and storage of signal parameters such as maximum amplitude, duration, rise time, and energy (RILEM Technical Committee 2010)
- (f) Storage of full transient wave forms
- (g) Autonomous recording and remote access via local area network (LAN) connection

4.1.2.4 Computer to store the recorded signals and parameters—A standard personal computer (PC) is usually sufficient to define data acquisition settings and host the analysis software. In some cases, the data acquisition hardware and computer are integrated in one unit.

4.1.2.5 Software to process and analyze the recorded signals and parameters—Commercially available AE systems are usually equipped with a basic software to collect, organize, and analyze AE data. Specific capabilities include:

- (a) Basic statistical analysis of extracted signal parameters such as AE activity (hit rates), amplitude distributions (b -value analysis), and parameter correlations (strain versus activity)
- (b) Forming of events based on hits recorded by multiple sensors within a network
- (c) Pattern recognition and neural network analysis (optional feature)
- (d) Estimation of two- or three-dimensional (2-D or 3-D) source locations (optional feature)
- (e) Estimation of source parameters using moment tensor inversion (advanced optional feature)

4.1.3 Method of deployment—A typical AE measurement process is illustrated in Fig. 4.1.3. The individual components of the system were described in detail in Section 4.1.2.1 and the entire process can be described in mathematical

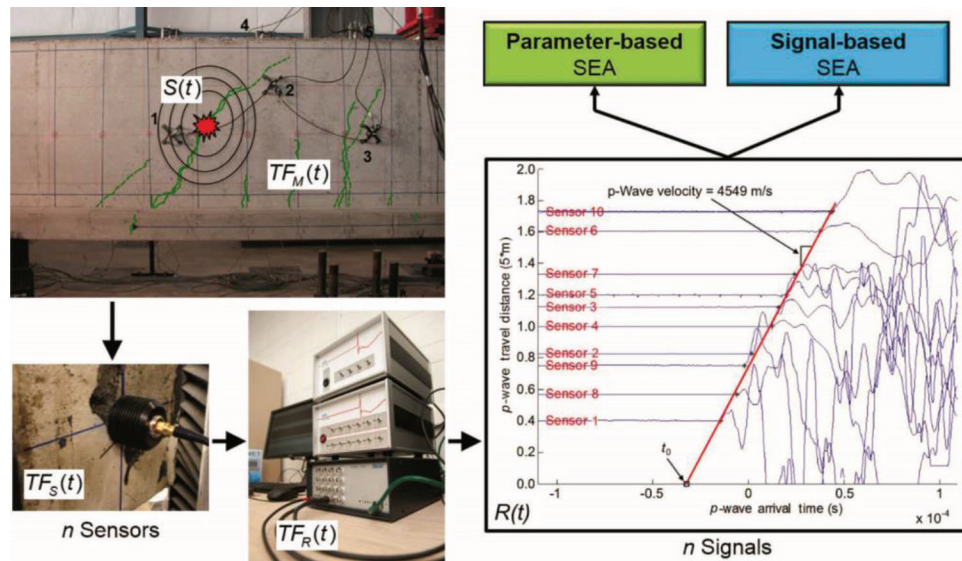


Fig. 4.1.3—Typical measurement chain for AE monitoring (adapted from Grosse and Schumacher [2013]).

Table 4.1.4—Comparison of analysis approaches

Category	Parameter-based (qualitative)	Signal-based (quantitative)
Requirements for AE system and instruments	Minimal; can theoretically be performed using a single sensor	High; sensor networks with at least six sensors required
Analytical procedures	Statistics computed from AE parameters, per signal (or sensor), simple forward methods	Complex nonlinear inverse methods using AE events
Results	Comparative (relative) analyses; goal is to reveal trends and patterns from large AE parameter sets	Absolute estimation of source location and characteristics; goal is to characterize every event
Strengths	Application simple and inexpensive; can give a fast empirical overview of temporal evolution of AE parameters; can be performed in real time; can cover a relatively large area with few sensors	Detailed results showing location and fracture-type of each AE source; often performed using post-processing, limited to local area in the vicinity of the anticipated fracture zone
Limitations	Limited to situations where the boundary conditions are mostly consistent (geometry, material properties, existing crack patterns); quality of recorded signals difficult to judge because sources are unknown; purely empirical relationships	Computationally expensive, complex procedures require high-quality signals; simplified assumptions (for example, homogeneous medium) may not be adequate; inversion can be difficult and unstable due to sparse information and noise
Applications	In-service structural load testing (short- and long-term)	Monitoring and characterization of fracture processes

ical form as a series of transfer functions convolved with the source signal

$$R(t) = S(t) \cdot TF_M(t) \cdot TF_S(t) \cdot TF_R(t) \quad (4.1.3)$$

where $R(t)$ is the recorded signal; $S(t)$ is the AE source signal; $TF_M(t)$ is the elastic Green's functions that describe the wave propagation (and theoretically need to include inhomogeneities and isotropic behavior) of the medium; $TF_S(t)$ is the sensor response characteristics; and $TF_R(t)$ is the characteristics of the data acquisition system (for example, used filters and response characteristics of amplifiers). Components with a flat frequency response that are provided by high-fidelity devices can be neglected—that is, their response functions can be set to a constant value of 1. In summary, the relationship in Eq. (4.1.3) highlights the need to carefully consider each component of the measurement process and how it affects the actual AE source signal. In other words,

what is displayed on the screen may only remotely reflect the actual source characteristics.

AE monitoring systems may be deployed over the short or the long term. Short-term applications include structural load testing or monitoring during routine bridge operations such as lifting or drawing of moveable bridges. Long-term deployments include applications during regular operations of bridges to collect in-service data.

4.1.4 Strengths and limitations—AE events result from fracture within a material medium—that is, these events represent the actual input signal (as opposed to, for example, ultrasonic testing [UT] where the input signal is created by the operator [ACI 228.2R]). The challenge is to interpret the signals recorded by the sensors because they are influenced by many factors, as described in Section 4.1.3. Concrete is a highly heterogeneous material and therefore complicates and interferes with the propagating stress wave. Signal attenuation due to scattering interaction with aggregate

and pores is significant (Landis and Shah 1995; Owino and Jacobs 1999).

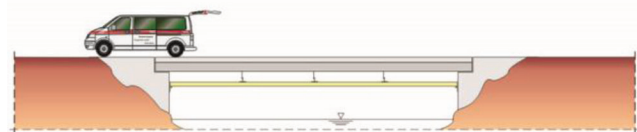
Strengths and limitations are also governed by the type of deployment and analysis approach that are used. There are two main approaches that can be taken to analyze AE data: qualitative (or parameter-based) and quantitative (or signal-based) (Mhamdi et al. 2015). Table 4.1.4 gives an overview of the two approaches and their strengths and limitations.

4.1.5 Availability and readiness for application in the field—Due to the limitations presented in Section 4.1.4, only qualitative approaches have so far been applied in the field. Many examples of short-term monitoring using qualitative analyses can be found in the literature (Ohtsu 1996; Golaski et al. 2002; Colombo et al. 2003; Ziehl and Lamanna 2003; Lovejoy 2006; Vogel et al. 2006; Suzuki et al. 2007; Schumacher et al. 2013; Anay et al. 2015). One interesting application developed at the University of Applied Sciences Bremen, Germany, involves a special vehicle, called BELFA, which is capable of applying a controllable load to

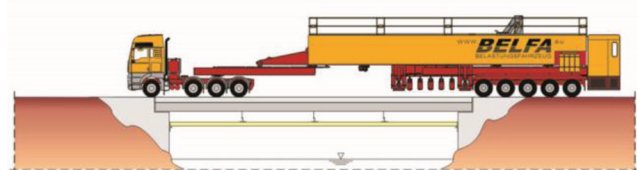
a full-scale structure using hydraulic actuators. This apparatus can be used for in-service load testing of bridges with a length of up to 35 m (115 ft) (Bretschneider et al. 2012). These short-term analyses can be extended to either SHM or short-term NDT of structures. During testing, a suite of sensors, including AE sensors, capture the bridge's response. The role of the AE sensors is to capture ongoing fracture and warn from potential overloading. The testing procedure is illustrated in Fig. 4.1.5.

4.1.6 Available standards and codes of practice—Currently, only a few recommendations or standards exist that provide guidance on the application of AE monitoring for concrete structures. The Japanese standard **NDIS 2421** represents one of the first recommendations and was mainly developed for AE testing of reinforced concrete structures (Ohtsu et al. 2002). The procedure involves application of loads with increasing magnitude to the structure and subsequent computation of two main parameters, called **LOAD** and **CALM** ratio. A recent RILEM standard defines basic AE terminology and parameters and refers to numerous ASNT, ASTM, EN, EWGAE, and the NDIS recommendations (RILEM Technical Committee 2010). To conclude, additional research work and a better understanding of the origin of AE signals is required before this method can be applied with confidence in the field.

Deployment of sensors on bridge



Deployment of BELFA test vehicle



Structural load test

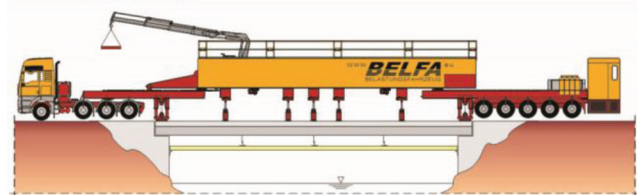


Fig. 4.1.5—Illustration of testing procedure with the BELFA vehicle (adapted from illustration retrieved from www.belfa.eu).

4.2—Conductive surface sensors

4.2.1 Physical basis and property inferred—Conductive surface sensors (CSSs), also referred to as sensing layers, sensing skins, surface sensors, and crack detection sensors, are electrically-conductive materials applied to the surface of structural materials or elements. When the concrete substrate is strained, a CSS is stretched and its electrical resistance increases, as illustrated in Fig. 4.2.1a. When the concrete cracks, a CSS ruptures, causing its electrical resistance to increase suddenly by orders of magnitude (Pour-Ghaz 2011). This change in electrical resistance can be measured at two or more externally applied electrodes and used to monitor strain and cracking in a structural element.

CSSs can be applied to structural elements that are electrically nonconductive such as concrete, reinforced concrete, and certain fiber-reinforced polymer (FRP) materials. A CSS can also be used to monitor cracking in a structural element that is electrically conductive if the monitored surface is

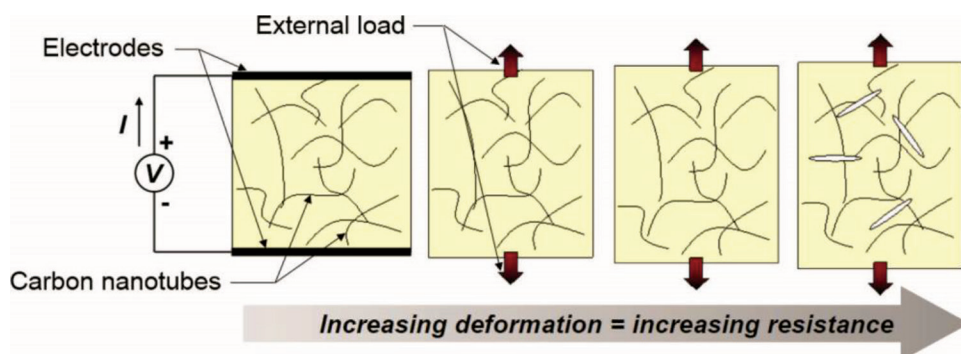


Fig. 4.2.1a—Fundamental principle illustrated on a carbon nanotube-based CSS (only two electrodes shown for simplicity). resistance measured at electrodes increases with increasing deformation (adapted from Schumacher and Thostenson 2014).



Fig. 4.2.1b—(a) Reinforced concrete pipe segment instrumented with copper metal foil-based one-dimensional CSS (Pour-Ghaz et al. 2011a); (b) restrained ring test instrumented with colloidal silver-based CSS. The response of the CSS is interrogated using radio frequency identification (RFID) technology (Pour-Ghaz et al. 2014).

first electrically isolated from the CSS—for example, by applying a nonconductive paint or epoxy layer.

CSSs can be produced and used in a variety of shapes tailored for specific applications. Figure 4.2.1b illustrates examples of applications of CSSs. Figure 4.2.1b(a) illustrates an instrumented reinforced concrete pipe segment with a copper metal foil CSS (Pour-Ghaz et al. 2011a). Figure 4.2.1b(b) illustrates a colloidal silver-based CSS applied to a restrained ring test specimen to detect the time of cracking of the concrete ring specimen; the response of the CSS in this setup is interrogated using radio frequency identification (RFID) tags (Pour-Ghaz et al. 2014).

Different measurement strategies can be used to monitor the response of a CSS. In one-dimensional (1-D) applications, electrical resistance is measured between two electrodes using a digital multi-meter (DMM) or electrical impedance spectroscopy (EIS) (Pour-Ghaz and Weiss 2011a,b; Pour-Ghaz et al. 2013). In two-dimensional (2-D) geometries, the use of multiple electrodes combined with advanced measurement techniques such as electrical resistance tomography (ERT) or electrical impedance tomography (EIT) can be used (Hou et al. 2007; Hou and Lynch 2009; Hallaji and Pour-Ghaz 2014; Hallaji et al. 2014; Loyola et al. 2013; Dai et al. 2016). ERT and EIT, however, require computational efforts for reconstruction of resistivity (or conductivity) distribution. Figure 4.2.1c(a) illustrates a 2-D copper-based CSS applied to a concrete substrate where ERT is used as a method of locating damage on the sensing skin (Hallaji et al. 2014). Figure 4.2.1c(b) illustrates a photograph of the CSS with highlighted cracks to aid visualization. Figure 4.2.1c(c) illustrates the results of quantitative electrical resistivity distribution reconstruction using ERT (Hallaji et al. 2014).

Different conductive materials have been previously used to develop CSSs, including thin copper metal foils (Pour-Ghaz and Weiss 2011a), colloidal silver paint (Raoufi et al. 2011), colloidal copper paint (Hallaji and Pour-Ghaz 2014), silver-coated copper particles (Hallaji et al. 2014), carbon-filled epoxies (Pour-Ghaz 2011), and carbon nanotube-based composites (Thostenson and Chou 2006; Hou et al. 2007; Loh et al. 2007; Hou and Lynch 2009). The advan-

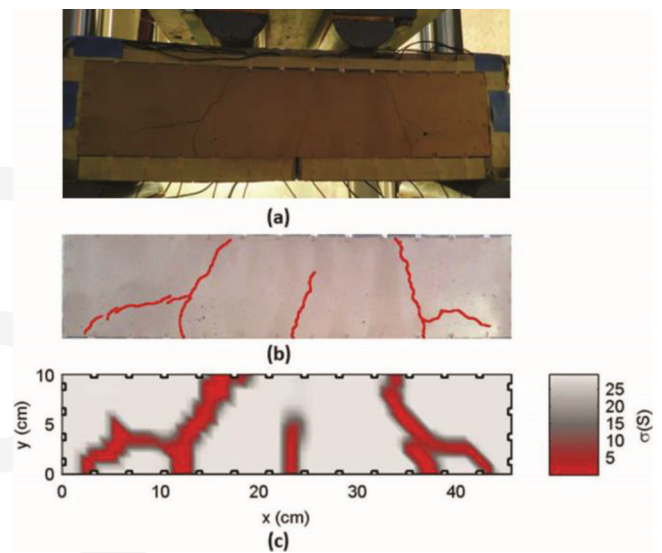


Fig. 4.2.1c—(a) Photograph of a two-dimensional copper-based CSS applied to a concrete beam substrate where ERT is used as a method of locating damage on the CSS; (b) highlighted cracks on the beam to aid visualization; and (c) results of electrical resistivity distribution reconstruction using ERT (Hallaji et al. 2014). (Note: 1 cm = 0.394 in.)

tages and limitations of different materials are discussed in detail in Pour-Ghaz et al. (2011b) and Loyola et al. (2013). Generally, colloidal paint-based sensors are more easily applied and have very low initial resistivity. These sensors are directly applied to the substrate in the form of paint and cure relatively quickly (a few minutes to a few hours, depending on the material and the methods of application) and, therefore, the installation time is comparatively short. Colloidal paint-based CSSs have higher resolution in crack detection as compared to metal foil-based conductive surface sensors. However, colloidal paint-based CSSs are not damage-tolerant and may be more easily damaged under service life conditions. Metal foil-based CSSs, on the other hand, are more robust sensors. Copper metal foil-based CSSs have been used, for example, in full-scale testing of buried concrete pipelines to investigate the modes of failure of buried segmental concrete pipelines (Pour-Ghaz et al.

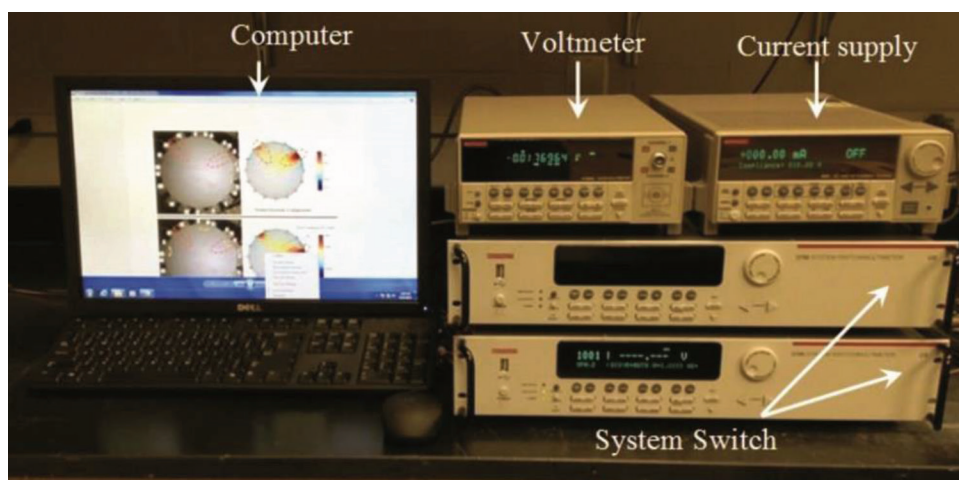


Fig. 4.2.2—Photograph of an electrical impedance tomography equipment set (courtesy of Materials and Sensor Development Laboratory, North Carolina State University) (Hallaji and Pour-Ghaz 2014).

2011a). These sensors performed well under field-simulating conditions. Carbon nanotube (CNT)-based sensors require the integration of carbon nanotubes into a nonconductive fabric, which then is applied to the substrate (Gao et al. 2010a,b; An et al. 2012). The advantage is that the fabric can be either nonstructural (for sensing only [Dai et al. 2014]), or structural (to form an integrated reinforcing/monitoring methodology [Schumacher and Thostenson 2014]).

The above-mentioned technologies are mainly based on measurement of electrical resistance or impedance. Another class of CSS are capacitive sensors that operate based on measurement of electrical capacitance (Kollosche et al. 2011). These sensors are mainly based on elastomeric materials.

4.2.2 Equipment needed—The choice of the equipment that can be used for monitoring the response of a 1-D CSS is dependent on the objective of the use. Different measurement strategies and equipment setups can be used. For tracing small changes in electrical resistance of a CSS, generally, the use of a highly accurate DMM (for example, with mV resolution) combined with a Wheatstone bridge circuit might be necessary (Pour-Ghaz 2011). For crack detection application, when the electrical resistance changes are expected to be relatively large compared to the initial electrical resistance of the sensor, only an accurate DMM (for example, with mV resolution) is generally needed to perform the measurements. To identify the crack location using a CSS, different measurement strategies such as parallel electrode arrangement is necessary, which requires the use of more advanced measurement systems such as EIS (Pour-Ghaz 2011; Pour-Ghaz and Weiss 2011a). To interrogate the response of multiple CSSs simultaneously, the use of frequency-selective circuits and EIS equipment is necessary (Pour-Ghaz and Weiss 2011b). The response of a CSS can also be interrogated wirelessly using radio frequency identification (RFID) technology where the use of RFID tags and RFID readers is necessary (Loh and Lynch 2009; Pour-Ghaz et al. 2014). In 2-D applications where the use of ERT is necessary for damage detection and localization, the use of ERT equipment is necessary (Hallaji and Pour-Ghaz

2014; Hallaji et al. 2014). Figure 4.2.2 shows a photograph of an ERT/EIT equipment set (Hallaji and Pour-Ghaz 2014).

4.2.3 Method of deployment—Various methods can be potentially used for the installation of a CSS, depending on the type of sensor and the environmental/operational conditions that the CSS will be exposed to. Generally, in harsh operation conditions, metal foil-based CSSs are more suitable as compared to paint-based CSSs because metal foil-based sensors are more tolerant to damage during installation and operation (Pour-Ghaz et al. 2011a). CSSs can also be coated to avoid damage during installation and operation or to avoid corrosion during the service life of the structure. Epoxy coating on metal foil-based CSSs installed on segmental concrete pipelines have been previously used by researchers to avoid damage to the CSS during the installation process and assembly of the pipeline (Pour-Ghaz et al. 2011a).

Paint-based CSSs can be directly applied to structural elements. Various methods, including deposition using brushes and spray, can be used for applying paint-based CSSs. In general, spray painting a CSS on a structure results in a more uniform electrical resistivity throughout the length of sensor. Before a CSS can be installed, some surface preparation is often required. In general, when the spray-painting method is used, the surface should be smooth and clean. Foil-based CSSs are generally more tolerant to roughness. Currently, no quantitative measure for surface preparation exists.

Both metal foil-based and paint-based CSSs can be used in 2-D sensing applications. In general, paint-based CSSs (both 1-D and 2-D) are more sensitive and detect cracks on concrete elements at an early stage. In 2-D applications where ERT measurements are used for detecting and locating cracks, initial uniform conductivity distribution provides better reconstruction results and is therefore preferred to applications by brush that may not provide uniform initial conductivity distribution (Hallaji et al. 2014).

Carbon nanotube-based CSSs consist of a nonconductive fabric that is bonded to the substrate using a suitable epoxy (Schumacher and Thostenson 2014; Dai et al. 2015). They

Table 4.2.4—Summary of strengths and limitations of CSSs

Strengths	(a) Sensitive to small strain changes as well as microdamage such as cracking (b) Distributed measurement (c) Materials and measurement systems relatively inexpensive (1-D sensing) (d) Can adhere to almost any geometry and shape as well as type of material (e) High flexibility where measurement points are placed and how many of them
Limitations	(a) Measurements can be directly related to strain only for the 1-D sensing case (b) Measurements are surface observations (c) 2-D sensing requires elaborate measurement systems and significant computational resources (d) Performance under real-world conditions have not been fully demonstrated

can be coated with a suitable epoxy to keep conductivity constant and protect the fabric from deteriorating.

4.2.4 Strengths and limitations—The main strength of CSSs is that they are very sensitive to small strain changes and the formation of microdamage such as cracking. Additionally, they allow for distributed sensing, meaning they are sensitive over the entire area, as defined between the electrodes, compared to quasi-point sensors such as strain gauges or accelerometers. This significantly increases the chances of capturing locations of high strain or damage initiation. CSSs are relatively inexpensive. These sensors can be adhered to literally any geometry and shape, permanently on new or existing structures for long-term monitoring. The interpretation of the response of these sensors is straightforward. The response of a CSS is interrogated by measuring its electrical resistance (in 1-D sensors) or by reconstructing the change in the electrical conductivity distribution (in 2-D sensing skins). In the case of 1-D sensing skins, the required equipment for monitoring the response of a CSS is rather inexpensive. In 1-D applications for crack detection, the length of the CSS can be rather large (several yards [meters]). Similarly, in 2-D applications for crack detection, the covered area using a CSS can be large; however, the minimum detectable crack length decreases with the size of a CSS. The dependence of the minimum detectable crack length to the size of the CSS is a function of many factors, such as the shape and the number of electrodes, and should be evaluated for each geometry (Hallaji and Pour-Ghaz 2014).

A major limitation of CSSs is that they can only detect strains and cracks at the surface of an instrumented structural element and are thus not sensitive to internal cracks and defects. Also, reconstruction of the conductivity distribution of 2-D CSSs requires elaborate measurement systems and significant computational resources. The strength that CSSs are flexible in their application and are highly customizable, depending on their application, also introduces a limitation: every CSS is unique and must be calibrated after it has been applied. The calibration of a CSS installed onto large or complex structures could be particularly challenging. In many cases, it may be impossible to relate measured resistance to strain directly, and only relative comparisons are possible. Finally, the performance under real-world environmental conditions and over the long term have not been fully demonstrated.

Table 4.2.4 summarizes the strengths and limitations of CSSs.

4.2.5 Availability and readiness for application in the field—Researchers have successfully developed and used

CSSs in different applications. Pour-Ghaz and Weiss (2011a) developed and used copper- and silver-based CSSs to detect cracking of concrete in restrained ring and restrained slab geometries. By modifying the arrangement of the electrodes and using CSSs, these researchers were able to detect the time and location of cracking (Pour-Ghaz and Weiss 2011a). CSSs were also used in full-scale testing of buried concrete pipelines to investigate the modes of failure of buried segmental concrete pipelines (Pour-Ghaz et al. 2011a). Raoufi et al. (2011) used CSSs in conjunction with acoustic emission monitoring to understand the effect of substrate debonding on the opening of the cracks of restrained concrete elements when shrinkage cracking occurs. Pour-Ghaz et al. (2011b) compared the CSS results with the time of visible cracking and a good correlation was observed. The authors used frequency-selective circuits (FSCs) to interrogate the response of multiple conductive surface sensors with a single electrical impedance spectroscopy measurement. More recently, radio frequency identification (RFID) technology was used in conjunction with these CSSs for wireless damage detection in concrete materials and reinforced concrete elements (Pour-Ghaz et al. 2014). Kollosche et al. (2011) also used surface materials to detect cracking by measuring the capacitance of elastomeric materials applied to the surface of concrete. Recently, Hallaji and Pour-Ghaz (2014) and Hallaji et al. (2014) extended into 2-D CSSs and used electrical resistance tomography to detect and localize the damage on these 2-D sensing skins. Despite the significant amount of research in this area, these sensors and their associated measurement technology are not currently commercially available.

4.2.6 Available standards and codes of practice—Currently, no standard method or recommended practices for application of conductive surface sensors and sensing skins to concrete structures are available.

4.3—Digital image correlation (DIC) technique

4.3.1 Physical basis and property inferred—Digital image correlation (DIC) is a non-contact, nondestructive optical technique capable of measuring two- and three-dimensional (2-D or 3-D) displacements of an element subjected to mechanical and/or thermal loading by comparing consecutive images recorded before and during the loading. These images can be acquired from numerous sources, including charge-coupled device (CCD) cameras, digital single-lens reflex (SLR) cameras, and point-and-shoot or even high-resolution video. These digital images consist of a high number of pixels, which collect exclusive color intensity

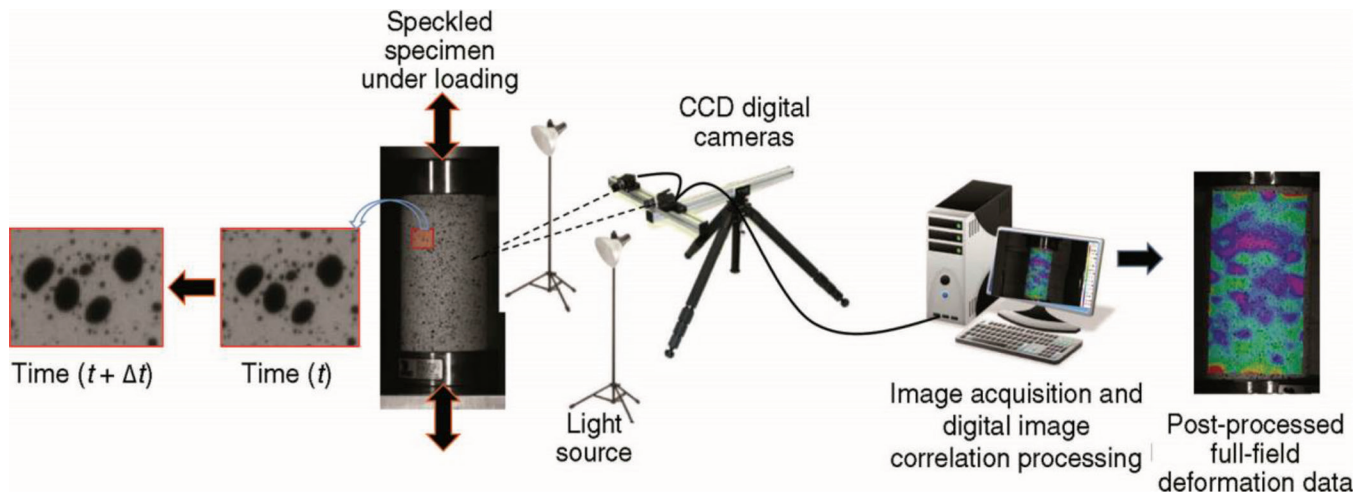


Fig. 4.3.1a—Schematic of digital image correlation concept (Harris 2014).

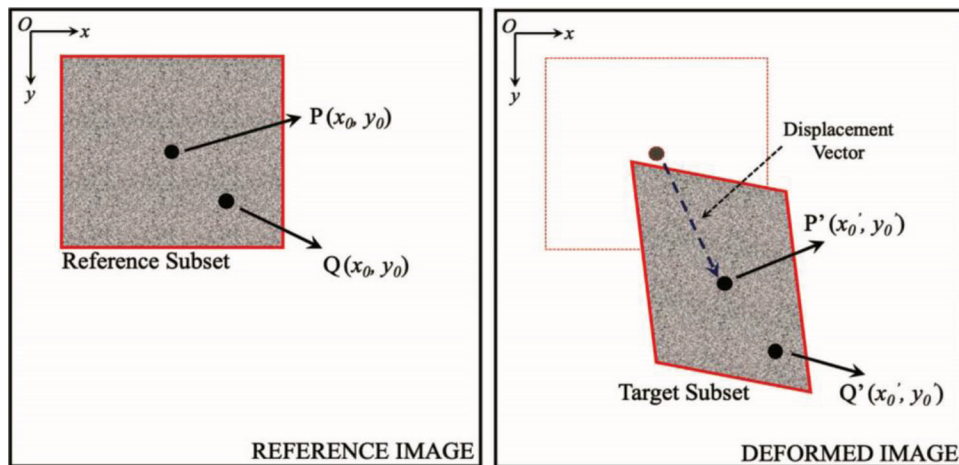


Fig. 4.3.1b—Basic concept of DIC (adapted from Pan et al. [2009]).

data of every point on the target surface. To mitigate correspondence issues associated with uniqueness of an individual pixel, images are typically divided into smaller regions or subsets (discretized areas with unique pixel features). Figure 4.3.1a (Harris 2014) shows a schematic illustration for the DIC concept derived from descriptions presented by Sutton et al. (2009) and Sutton (2013). The DIC technique derives continuous incremental displacement data by tracking or correlating the movement of unique subsets from image to image and interpolating deformation within the subsets, as shown in Figure 4.3.1a. The incremental displacements can be transformed later into strain via post-processing.

The concept of digital image correlation works for both 2-D and 3-D measurements. For 2-D, in-plan, displacement measurements, a single camera or a series of uncorrelated cameras can be used to measure deformation in a plane parallel to the camera's focal plane (for example, x-y plane). For 2-D DIC, a known distance between any two points on the object is used to establish the scaling factor that provides dimensionality to the deformations (that is, allows measurements in pixels to be converted to physical measurements). However, if the object will develop out-of-plane displacement, this method requires that the camera

remains perpendicular to the plane of measurement, otherwise the out-of-plane movement can lead to significant errors unless accounted for (Hoult et al. 2013). On the other hand, 3-D DIC uses stereo pairs of cameras to incorporate out-of-plane deformation. This approach does not require the same planarity of the 2-D approach but must integrate a calibration using the intrinsic and extrinsic parameters of the cameras to produce positioning parameters and yield absolute deformation measurements. The differences between subsets in consecutive images are interpolated, similar to the finite element method, providing the ability to describe the full-field deformation over the specimen surface. The basic concepts of DIC are summarized in Fig. 4.3.1b.

The resolution of this approach is controlled by the camera and lens selection in addition to the image-matching (correlation) algorithms used. Typically, a contrasting speckle pattern (for example, a random white and black spray or dot pattern) is applied to the surface of the area of interest. This pattern is used within the correlation algorithm to distinguish, map, and track the unique patterns of pixels within a subset region. An external light source is often used to create uniform illumination across the surface area of interest. In general, the accuracy of the DIC technique is on the order

of a fraction of a pixel (Pan et al. 2009; Sutton 2008; Sutton et al. 1991; Peters and Ranson 1981). Commercial vendors have suggested that the feasible strain measurement range varies between 0.001 and 300+ percent (Trilion Quality Systems 2015) with achievable resolutions on the order of 1/100,000 of the field of view for in-plane measurements and 1/50,000 of the field of view for out-of-plane measurements (Correlated Solutions, Inc. 2015). A study on error assessment in stereo-based deformations by Ke et al. (2011) demonstrates the feasibility and accuracy of the predictive formulas for estimating the uncertainty in the stereo-based deformation measurements. Suitable algorithms to minimize lens distortion effects and variations in lighting can be found in other references (Pan et al. 2013; Yoneyama et al. 2006), but it should be noted that both setups use a correlation function to map the optimal displacements of pixels. For example, Eq. (4.3.1) describes a correlation algorithm based on the sum of squared differences (SSD) of the pixel values used for optimal correlation estimates (Sutton et al. 2009). A minimization of this correlation function is used to evaluate the likeness of pixel patterns within a subset region across successive images as the surface is deformed (u and v). A low correlation function indicates a match between the patterns of successive images. With the pixel patterns correlated, the displacements can be mapped and described continuously across the surface via a shape function. The SSD correlation function (Eq. (4.3.1)) is one example used in DIC, but many others are available in the literature (Tong 2005; Pan et al. 2010)

$$C(\bar{x}, \bar{y}, u, v) = \frac{1}{\sum_{i,j=-n/2}^{n/2}} \left(I(\bar{x} + i, \bar{y} + j) - I^*(\bar{x} + u + i, \bar{y} + v + j) \right)^2 \quad (4.3.1)$$

where C is the correlation function; n is subset size; I is image before motion; I^* is image after motion; u and v are displacements (disparity) in the x - and y -directions, respectively; x and y are pixel coordinates in reference image; $(\bar{x} + i, \bar{y} + j)$ is pixel value at $(x + i, y + j)$; and $(\bar{x} + u + i, \bar{y} + v + j)$ is pixel value at $(x + u + i, y + v + j)$.

4.3.2 Equipment needed—Digital image correlation in a general sense requires a very basic configuration that consists of specimen preparation and camera setup; however, sample preparations and, particularly, the patterning operations, can be rather time-consuming (Helm 2008; Ghorbani et al. 2015a). If the natural texture of a specimen surface does not provide a high enough variation in the gray level (contrast), the surface should be prepared by applying artificial speckle patterns to establish a unique and random pattern with high contrast. To this end, different techniques may be enlisted, such as white and black spray painting, using flexible stencils with optimized speckle patterns, and stamping (Helm 2008; Sutton et al. 2009; Ghorbani et al. 2015a). A white light source or natural light can be used for illumination during loading; thus, it is suitable for both laboratory and field applications (Pan et al. 2009). It is important to avoid excessive heating, as it may greatly affect DIC measure-

ments; for this reason, LED lamps may be a preferable option as a lighting source.

Therefore, the equipment and software needed for DIC can be divided into three parts: first, the surface preparation; second, the data acquisition “photos”; and third, the data processing. The basic equipment needed for the data acquisition and post-processing is listed in the following sections.

4.3.2.1 Surface preparation

(a) **Pattern application:** Flat paint in white and black is often used to apply the unique speckle pattern. There are various other methods described in literature (Helm 2008; Sutton et al. 2009; Ghorbani et al. 2015a) to apply a unique speckle pattern on the object (for example, stencils and markers), but these will not be fully discussed in this chapter. Typically, the speckle pattern should be selected such that speckles range between two and five pixels in diameter (Zhou and Goodson 2001) and tailoring the subset size such that individual subsets contain approximately 3 x 3 speckles (Sutton et al. 2009). In addition, the user is cautioned to ensure the pixels are appropriately sized to ensure that aliasing does not occur.

(b) **Lighting:** Diffused lighting is needed to evenly illuminate the region of interest. In outdoor applications, it may be necessary to shield the object from direct sunlight and ensure that the images are not oversaturated with light or washed out. Examples of these outdoor applications are described by Ghorbani et al. (2015b) and Gencturk et al. (2014).

4.3.2.2 Data acquisition

(a) **Cameras:** Modern advances in digital camera resolution and frame rates (SLR, point-and-shoot, and CCD) allow DIC to capture images with sufficient detail to determine the physical position changes between two or more images (Pebble 2011). As an example, Harris (2014) used two CCD digital cameras with a resolution of 5 megapixels and 15 frames per second in a small-scale concrete flexural test for 3-D DIC measurements of strain. In contrast, Ghrib et al. (2012) used a single DSLR digital camera with a resolution of 12 megapixels for 2-D DIC measurements of deflection of large-scale RC beams. Figure 4.3.2.2 provides an illustration of the experimental setups for both laboratory experiments.

(b) **Tripod:** Movement of the cameras during a test will produce inaccurate results. Minimizing motion is extremely important when using stereo-paired cameras to prevent disturbing the calibration of the system. A sturdy mounting surface, such as a tripod, is necessary to prevent cameras from moving during the test.

(c) **Camera lens:** Each setup is dependent on the distance of the cameras from the object and the focal length of the selected lenses. Once the setup is calibrated for a given focal length, the camera focus must not be altered. The trade-off between distance between the cameras and the object and the required field of view will determine the acceptable focal length of the system.

(d) **Calibration targets:** Targets with standardized reference positions are used to calibrate 3-D DIC systems. For 2-D DIC, a known distance on the object or within the field of view must be used to provide scale of the measurement.

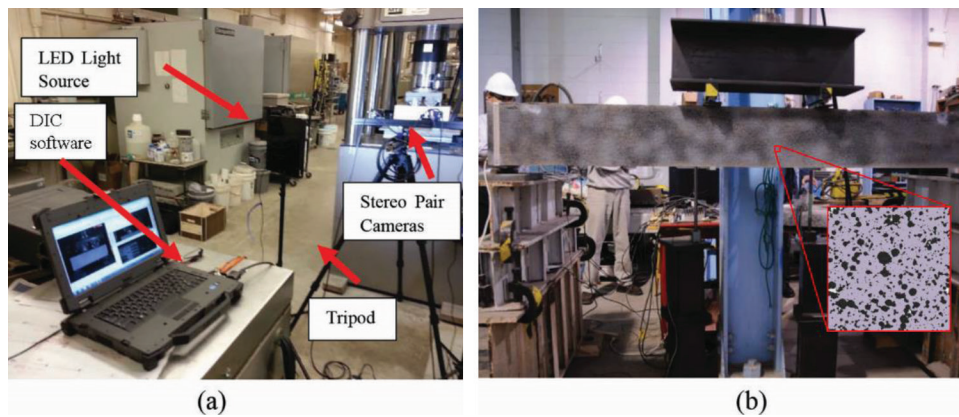


Fig. 4.3.2.2—Typical DIC laboratory experiments: (a) setup and equipment for small-scale concrete flexural testing (Harris 2014); and (b) large-scale concrete specimen with surface speckle pattern (Ghrib et al. 2014). The inset shows a close-up view of the speckle pattern.

(e) **Cable to connect cameras to computer:** High-transfer-rate cables such as FireWire 800 cables are often used to transfer the image data to a computer; however, images can also be stored locally on the camera and transferred later for post-processing. Regardless of the storage means, it is essential to ensure that the data transfer and/or storage rate is sufficient for the system being set up.

(f) **Computer with sufficient storage speed and capacity:** For high-speed image acquisition and real-time analysis schemes, ensure that the write speed of the hard drive is fast enough for the amount of data being transferred and that the hard drive has the necessary storage capacity for the job.

(g) **Data acquisition (or image acquisition) software:** Image acquisition software can be used to streamline the image collection process. Commercial systems are available that can simultaneously collect analog data while capturing images. If automated acquisition is used, a proper naming convention should be used to catalog and organize the images collected. Proper file naming is needed to make sure that the proper images are paired for correlation.

4.3.2.3 Data processing—Digital image correlation software for analyzing data: Numerous commercial vendors provide software that is capable of performing digital image correlation. Examples include Correlated Solutions, Inc., LaVision, GOM, and Trilion. In addition to these commercial systems, numerous user-developed DIC programs have been developed, many of which are available as freeware. During data processing, a correlation criterion or image matching algorithm should be defined in advance before correlation analysis to evaluate the similarity degree between the reference subset and the deformed subset (Pan et al. 2009). Although different definitions of correlation criteria can be found in the literature, these correlation criteria can be categorized into four groups:

- (a) Cross correlation (CC)
- (b) Sum of absolute difference (SAD)
- (c) Sum-squared difference (SSD)
- (d) Parametric sum-squared difference (PSSD) criterion (Pan et al. 2010)

The algorithm should extract a large number of features and these features are highly distinctive, which allows a single feature to be correctly matched with high probability (Praveen and Dasu 2015). There are numerous free-ware programs available for use such as the Scale Invariant Feature Transform (SIFT) model (Lowe 2004).

4.3.3 Method of deployment—The integration of DIC with traditional structural health monitoring frameworks lies in the capabilities of the technology to provide the full-field deformation response of the structure under load or within the operating environment. This comprehensive measurement technique has the potential to be applied on the global scale for measurements such as deflection, rotation, strain, and/or vibration, but also on the local scale for evaluating features such as fatigue-sensitive details, stress concentrations, or unexpected deformations. It is envisioned that this technology could be deployed as a real-time measurement tool, but also as a temporal measurement strategy. The anticipated benefit of this technology to the end-user would be a data-rich physical description of the structural response from which to make performance-based decisions. For applications in concrete and masonry, DIC can also be used to accurately measure displacements, interface slip, and map cracks and crack propagation (Ghorbani et al. 2015a). In addition, the non-contact, full-field nature of the technique allows for the measurement of average strains across arbitrary cracked sections, an application which cannot be achieved using traditional strain gauges (Dutton et al. 2014).

Deployment of DIC varies depending on the application. In general, the application of the DIC method for 2-D deformation measurements is carried out in five main basic steps:

1. **Specimen preparation:** applying the speckle pattern to the object surface
2. **Camera setup:** mounting on tripod and focusing the cameras on the area(s) of interest
3. **Imaging:** capturing the images as the area(s) or objects(s) move or deform under load
4. **Post-processing:** analyzing the acquired images using a computer program to obtain the desired displacement and strain information



Fig. 4.3.3.1a—Stereo-paired camera setup used during live load testing (Harris 2014).

For a 3-D DIC setup, an additional camera calibration is necessary before the previously described image capture task.

4.3.3.1 Example deployment—Figures 4.3.3.1a and 4.3.3.1b illustrate some of the previously described tasks used for deploying DIC in the field (Harris 2014). The application demonstrates the setup of 3-D DIC for measuring the deflection of an exterior bridge girder during a live load test, which includes the stereo-paired camera configuration and speckle-patterned target card mounted to the structure. The bridge being tested spanned 12.8 m (42 ft) over a water feature, making traditional deflection measurements challenging. To capture the deflection, the camera system was set up on one of the stream banks to image the girder and a speckle pattern target was mounted to the bottom of the exterior girder at midspan. For this example, 3-D DIC was used because of the challenge associated with establishing planarity between the target and cameras. The setup for measurement of global strain is generally the same, with the main difference being that the speckle pattern needs be applied directly to the area(s) or object(s) of interest. During the setup, the apertures of the cameras/lenses were fully opened to focus the lens on the target and then adjusted to let in the proper light for the given lighting conditions. After the system was focused, a series of calibration images were collected to determine intrinsic and extrinsic parameters of the cameras and positioning parameters for the determination of absolute deformation measurements. It is good practice to analyze the calibration images before the test to ensure proper system setup. Under load, the target deforms rigidly along with the exterior deformation and images were captured during the sequence of loading.

4.3.4 Strengths and limitations—A strength of DIC is that it provides full-field data in a non-contact manner. In addition, use of DIC does not require *a priori* knowledge about locations of interest (for example, high-stress regions and locations with cracking potential). As long as the area of interest is within the field of view, these areas can be evaluated during post-processing. DIC enables one to obtain full-

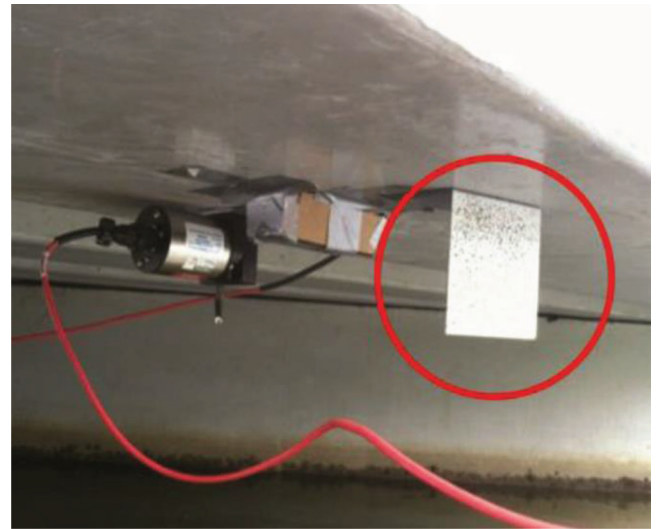


Fig. 4.3.3.1b—Speckle target mounted to girder bottom flange for measurement of deflections during live load testing (Harris 2014).

field strain measurements in contrast to traditional measurement techniques (for example, strain gauges), which are typically local. DIC can capture large strains and significant rigid body motion as long as the specimen remains in the field of view of the cameras. Lee et al. (2012) asserted that “the 2D-DIC strain measurement has the potential to provide the same strain accuracy as foil and vibrating-wire strain gauges for materials experiencing homogeneous strain.” When compared to traditional emplaced deflection measurement methods, the DIC setup is much faster and less labor-intensive. Typical setup time for an individual location is approximately 1 hour, depending on the number of cameras being used. The application of the speckle pattern can be done with spray paint. In addition, Wilbur (2011) concluded that DIC can be used for a wide range of object sizes as, compared to other techniques, it has straightforward procedures, high accuracy, uses off-the-shelf equipment, and specimens required minimal preparation before testing. As noted in Section 4.3.1, the resolution of DIC is related to the relationship between camera/lens parameters and experimental setup, but accuracies in literature have been reported to be on the order of 0.02 pixels (Wilbur 2011; Küntz et al. 2006; Yoneyama et al. 2007; Pan et al. 2009).

The biggest limitation with respect to the application of DIC in an SHM framework is that the technique requires a line of sight between the camera and the area of interest. The technique is also limited to surface measurements when traditional optical images are used and there is a limit to the resolution that must be considered when evaluating civil engineering structures that often experience low levels of strain. In addition, acquiring images in natural light can be challenging because the system is sensitive to the fluctuations in ambient light or inconsistencies due to reflections. As with most scientific equipment, the cameras are sensitive and must be handled with care to avoid damage.

Table 4.3.4 summarizes the strengths and limitations of the DIC technique.

Table 4.3.4—Strengths and limitations of the DIC technique

Strengths	(a) Provides full-field deformation measurements (b) Non-contact application (c) Able to capture large deformations as well as rigid body motions (d) Setup faster and less labor-intensive compared to traditional sensors (e) Under ideal circumstances, accuracy similar to traditional sensors can be achieved
Limitations	(a) Requires direct line of sight at all times to perform measurements (b) Limited to surface measurements (c) Accuracy limited by image resolution (d) Sensitivity to fluctuations in lighting conditions

4.3.5 Availability and readiness for application in the field—There are numerous commercial and research-developed DIC systems being used in the field. Systems may be customized to meet the specific demands of the user. The practicality depends on the constraints of the test and the test environment. For example, some locations have poor lines of sight and, therefore, this method for analyzing deformation may be inappropriate. This technology is gaining significant attention in the field of civil engineering, but many of the benefits have yet to be realized. Some representative field and large-scale applications of DIC are summarized in the following paragraph.

Küntz et al. (2006) used DIC to study the in-place behavior of a shear crack in a 23 m (75.5 ft) long reinforced concrete (RC) bridge girder during a static load test. The obtained resolution was on the order of 10 μm (4×10^{-4} in.). Yoneyama et al. (2007) also used DIC to monitor the deflection of 14.8 m (48.6 ft) long bridge girders during a static load test in the field. The standard deviation of the difference between the deflection measured by DIC and traditional displacement transducers during field load test on a 14.8 m (48.6 ft) span steel-girder bridge was approximately 0.31 mm (0.012 in.), which was equivalent to approximately 0.06 pixel. Destrebecq et al. (2011) studied the mechanical properties and structural behavior (deflection and cracking progress) of a 7.0 m (23.0 ft) long concrete beam after 25 years in service in an industrial environment. Easy crack detection and measurements are reported as main advantages of the DIC. A resolution of approximately 0.01 mm (4×10^{-4} in.) was achieved for a length-to-pixel ratio of 0.725 mm/pixel (0.029 in./pixel). Ghrib et al. (2014) deployed DIC to measure the full deflection profile along RC concrete beams tested in the laboratory. Gencturk et al. (2014) used the DIC technique to obtain full 3-D displacement and strain fields of 7.62 m (25 ft) long prestressed girders. It was concluded that the DIC technique provides very accurate and detailed information such as the in- and out-of-plane strains as well as critical tensile and compressive strains, which are not possible to obtain using conventional techniques. Ghorbani et al. (2015b) used DIC to investigate the progressive change of the deformed shape of roof asphalt shingles subjected to high wind loads up to failure.

4.3.6 Available standards and codes of practice—Numerous articles have been written about the use of DIC; however, for the most part, standardization is nonexistent. ASTM E2208 is one of the few standards available and relevant to DIC users. Its purpose is to help users understand the

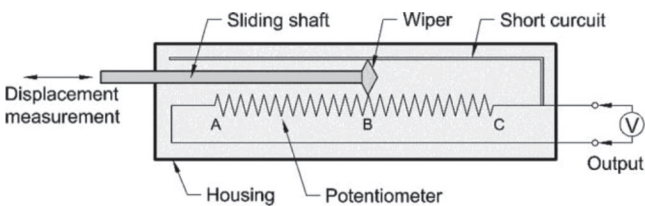


Fig. 4.4.1.1a—Schematic of a linear potentiometer.

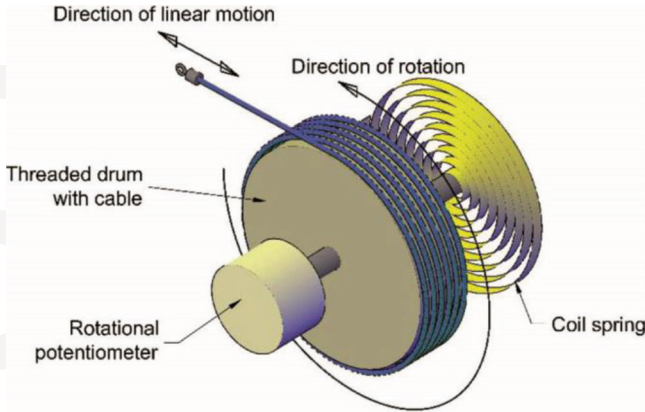


Fig. 4.4.1.1b—Schematic of draw-wire displacement sensor.

potential limitations in accuracy of non-contact systems for measuring strain.

4.4—Displacement sensors

Surface displacement may be measured using a wide variety of contact and non-contact methods. This section focuses on the two most common contact sensors: potentiometers and linear variable differential transformers (LVDTs).

4.4.1 Physical basis and property inferred

4.4.1.1 Potentiometers—A potentiometer, commonly known as a pot, is a resistive sensor used to measure translational and rotational displacements. It consists of an electromechanical device with an electrically conductive wiper that slides against a fixed resistive element according to the position or angle of an external shaft (Fig. 4.4.1.1a). Electrically, the resistive element is divided at the point of wiper contact. To measure displacements, a potentiometer is typically wired in a voltage divider configuration (Antonelli et al. 1999). The potentiometer outputs an analog voltage proportional to the position of the wiper and, therefore, proportional to the axial position of the shaft.

A string potentiometer, also known as a draw-wire sensor, is a form of potentiometer; refer to Fig. 4.4.1.1b for a sche-

matic representation. The wire clip at the end of the cable is attached to the desired measurement point on the monitored structure. As the structure moves, the cable unwinds the drum. A voltage signal that is proportional to the rotation of the drum—and hence the displacement of the end of the cable—is output by the potentiometer. A coil spring element ensures that the cable is in tension when extended and rewinds when released. This assembly will typically be housed in an aluminum or plastic case.

4.4.1.2 Linear variable differential transformers (LVDTs)—The linear variable differential transformer (LVDT) is an electric inductive transformer used for measuring linear displacement. A counterpart to this device used to measure rotary displacement is called a rotary variable differential transformer (RVDT). In this section, discussion will focus on the LVDT, though both sensors follow similar principles.

A schematic of a typical LVDT unit is shown in Fig. 4.4.1.2. The LVDT unit consists of two sets of wires, the primary and secondary annular windings, wrapped around a solid cylindrical core that is allowed to move axially. Constant amplitude AC current is supplied to the primary windings to set up a magnetic field in their center; the cylindrical core interacts with this field to create secondary fields that in turn create current in the secondary windings. The axial position of the central core affects the current generated in the secondary windings to create an output voltage. The windings are designed such that the output voltage is zero when the central core is positioned at the central axial location within the windings; movement of the core away from the center linearly increases the output voltage. The signal from the secondary coils is then further processed to provide an output value that is directly proportional with axial core cylinder movement while retaining positive or negative position information with respect to the central zero point.

4.4.2 Equipment needed—A typical use of displacement sensors is to measure the displacement of an object with respect to a fixed position, such as the opening of an expansion joint or central deflection of a flexural member. Sensor selection should consider the operating temperature, humidity, sensor robustness, and life cycle; these will vary by the sensor type and manufacturer per provided data sheets. At a minimum, the following equipment is needed to measure displacement for the two considered sensor types:

- (a) Potentiometers
 - i. A continuous voltage source to power the potentiometer
 - ii. A DAQ system to record the measured displacement
 - iii. External wires
- (b) Linear variable differential transformers (LVDTs)
 - i. A continuous voltage source to power the LVDT
 - ii. A signal amplifier/conditioner to provide the output data
 - iii. A DAQ system to record the measured displacement
 - iv. External wires

4.4.3 Method of deployment

4.4.3.1 Potentiometers—Potentiometers require that the housing be mounted to one surface or apparatus, and the shaft or string be connected to the measured object. Fundamen-

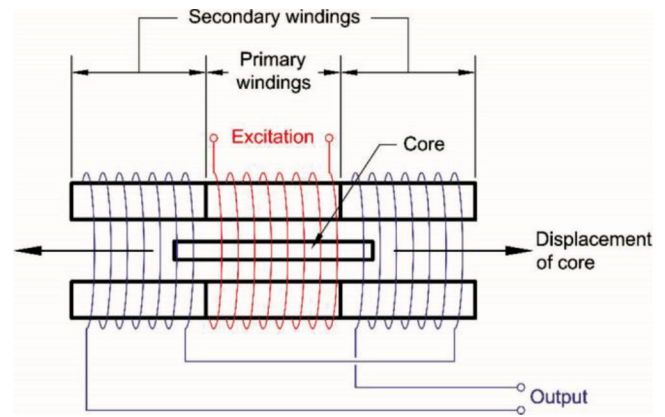


Fig. 4.4.1.2—Schematic of linear variable differential transformer.

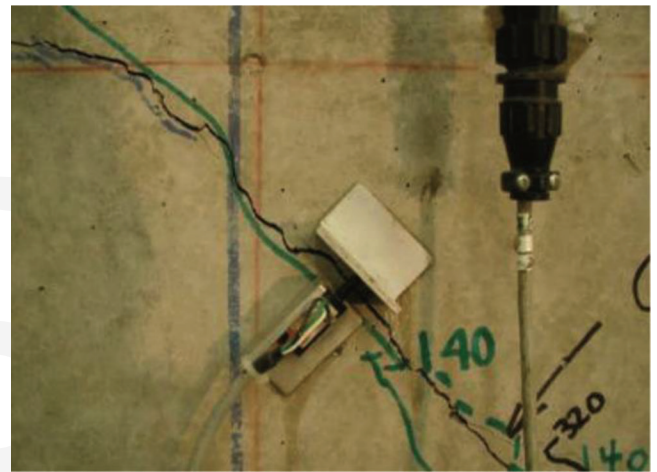


Fig. 4.4.3.1—Deployment of potentiometer on a large-scale concrete girder (Schumacher 2008).

tally, these sensors measure a change in distance between the housing and the end of the shaft or spring, which may limit the types of displacement measurable by these devices. Figure 4.4.3.1 shows a sample deployment of a potentiometer measuring crack opening on a concrete girder. The plastic housing of the potentiometer was attached to the surface of the concrete girder, and the shaft was pressed against a small angle attached to the girder on the other side of the crack to measure the crack opening.

4.4.3.2 Linear variable differential transformers (LVDTs)—Similar to potentiometers, LVDTs also measure the relative motion between the surface on which the housing is mounted and the end of the shaft. Because LVDTs rely on electromagnetic phenomena, they should be positioned away from or suitably shielded from other magnetic field sources such as electric motors and permanent magnets.

A sample installation for measuring girder displacements using an LVDT is shown in Fig. 4.4.3.2. A taut stainless steel aircraft reference cable was connected to the girder supports or end diaphragms to provide a reference base. The cable was mounted using angles and tightened by a turnbuckle. Additionally, creep and temperature effects on the cables were accommodated by attaching a spring to the cable to maintain proper tension. The LVDT was attached to the

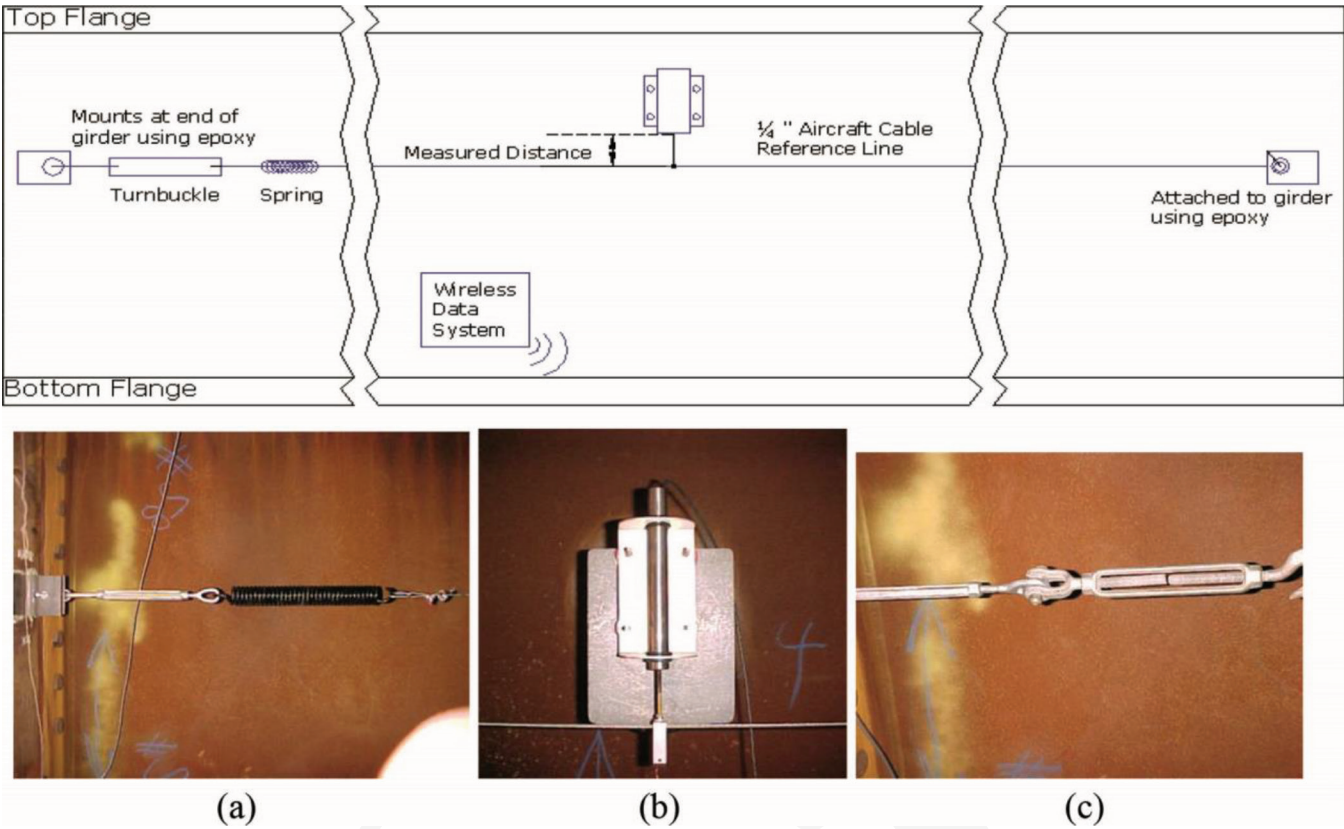


Fig. 4.4.3.2—LVDT cable system: (a) heavy-duty spring; (b) LVDT; and (c) turnbuckle (Nassif et al. 2005).

Table 4.4.4—Strengths and limitations of displacement sensors

	Strengths	Limitations
Potentiometers	(a) Easy to use (b) Inexpensive (c) Large measurement range (d) High accuracy	(a) Frictional loading: Wiper contacts with potentiometer element to produce signal (b) Inertial loading: Not suitable for dynamic measurement (c) May sustain damage if cable or shaft is quickly released
LVDT	(a) Frictionless measurement: Core does not contact windings and, therefore, allows for infinite resolution and very long mechanical life (b) No mechanical hysteresis (c) Environmentally robust (d) Fast dynamic response: They are the best solution in case of dynamic analysis (e) Insensitive to radial movement (f) High accuracy (below 0.01 mm [4×10^{-4} in.])	(a) Temperature changes may cause both scaling and zero-shift errors (b) Limited measurement range (c) Not perfectly linear when core is far from the central zero position (d) Due to small variations in the windings, a small residual voltage may appear at the output when the coil is in the central zero position

girder using a telescopic laboratory platform mounted on the centerline of the web of the selected girder with high-strength epoxy. Similar setups may be adapted for linear or string potentiometers.

4.4.4 Strengths and limitations—The strengths and limitations of the two types of displacement sensors are presented in Table 4.4.4.

4.4.5 Availability and readiness for application in the field

4.4.5.1 Potentiometers—Potentiometers (linear and string) are a mature technology available in a wide variety of different models and versions with variable cost, precision, range, and linearity.

4.4.5.2 Linear variable differential transformers (LVDTs)—LVDTs are widely available on the market. Typical LVDTs run at 3 to 15 V, can measure from 0.1 to

75 mm (4×10^{-3} to 3 in.), and may collect data at frequencies ranging from 60 to 20,000 Hz.

4.4.6 Available standards and codes of practice

4.4.6.1 Potentiometers—No available codes.

4.4.6.2 Linear variable differential transformers (LVDTs)—No available codes.

4.5—Fiber-optic sensors (FOSs)

4.5.1 Physical basis and property inferred

4.5.1.1 Structural property inferred—A large spectrum of parameters can be monitored using fiber-optic-sensing elements such as displacement (Section 4.4), strain (Section 4.8), inclination (tilt; Section 4.9), acceleration (Section 4.10), humidity (Section 5.3), pH (Section 5.5), and temperature (Section 5.7). However, compared with other sensors,

tional commercially available technologies that can be used to monitor the same parameters, FOSs offer two types of strain sensors that are unique to fiber-optic technologies: long-gauge and truly distributed sensors. The former enables global large-scale monitoring (with respect to a given structure), while the latter enables integrity monitoring, thus allowing the affordable instrumentation of large areas of civil structures and infrastructure. Thus, the scope of this document is limited to FOS for the measurement of strain. The methods for FOS strain measurement are proven to be suitable for performance evaluation and damage detection and characterization (localization, quantification and, to a certain extent, prognosis) in concrete structures in real on-site conditions. More information regarding strain as a structural property can be found in Section 4.8.

4.5.1.2 Physical basis of fiber-optic sensors: general considerations—An optical fiber is commonly made of fused silica and was originally designed for transmitting information via light over long distances in the telecommunications industry. The sensing fiber consists of three main components: core, cladding, and coating; refer to Fig. 4.5.1.2a.

The core transmits the light and for single-mode fibers, its diameter ranges between 5 to 10 μm (2×10^{-4} to 4×10^{-4} in.). For multi-mode fibers, the core diameter is approximately 50 μm (2×10^{-3} in.). The core is surrounded by cladding,

which is also made of silica but with a refractive index that is different from that of the core. Its external diameter is 125 μm (5×10^{-3} in.) and its purpose is to minimize the optical (light) losses and to physically support the core. Finally, the coating protects the fiber from the environment and reduces its fragility. Typically, the coating is made of acrylate (external diameter of 250 μm [0.01 in.]) or polyimide (external diameter of approximately 145 μm [6×10^{-3} in.]). Other, less typical coatings such as aluminum, carbon, and gold are also available, depending on the application and the required robustness. The wavelength ranges of light propagating through the optical fiber are approximately 1310 and 1550 nm (5.16×10^{-5} to 6.10×10^{-5} in.) for single-mode fibers and approximately 850 and 1300 nm (3.35×10^{-5} to 5.12×10^{-5} in.) for multi-mode fibers. Besides silica, polymer-based optical fibers exist as well, but their applications for sensing purposes are still at the research level and therefore are beyond the scope of this section.

There are several physical principles used to create sensing elements in FOS:

- (a) Michelson and Mach Zehnder Interferometers (SOFO – acronym derived from the French for ‘structural monitoring by optical fibres’ [surveillance d’ouvrages par fibres optiques])
- (b) fiber Bragg gratings (FBGs)
- (c) Brillouin scattering (BOTDA – Brillouin Optical Time Domain Analysis and BOTDR – Brillouin Optical Time Domain Reflectometry)
- (d) Rayleigh scattering
- (e) Extrinsic Fabry-Perot Interferometer (EFPI)
- (f) Intensity-based (micro-bending) sensing elements
- (g) Raman scattering

The first four are the most popular and were successfully applied in long-term concrete structure monitoring; thus, they are briefly presented in the following sections. EFPI was also frequently applied (historically it is the first

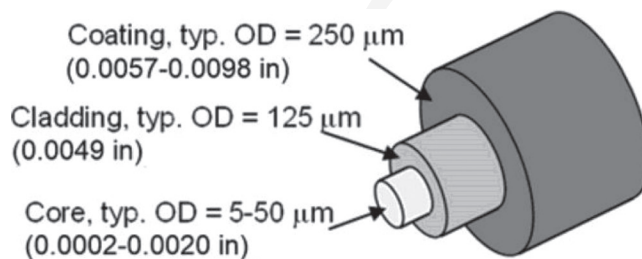


Fig. 4.5.1.2a—Components of standard optical fiber (Glisic 2009).

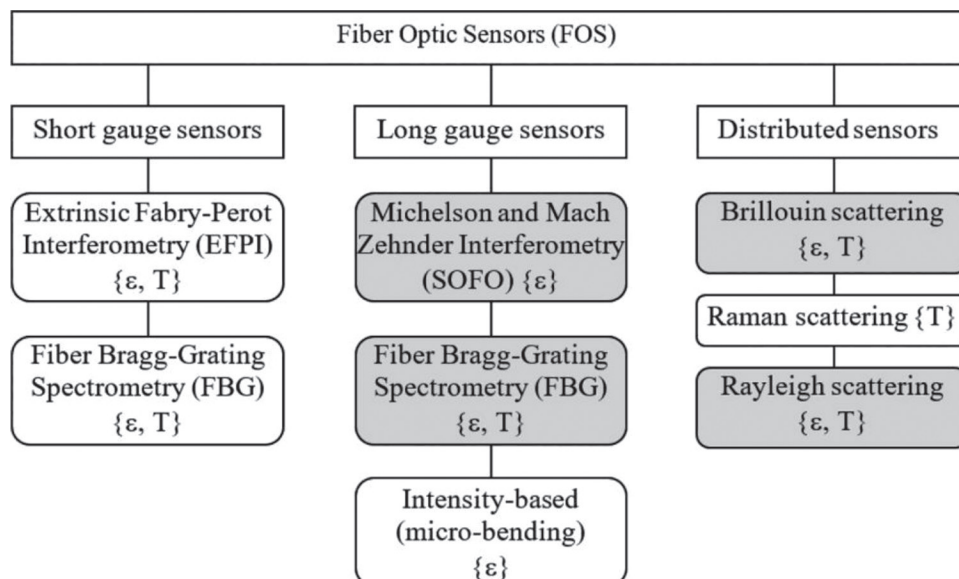


Fig. 4.5.1.2b—Classification of strain FOS based on their gauge length and functional principles. Shaded fields indicate technologies presented in this document (Glisic 2009).

Table 4.5.1.2a—Best performance of discrete strain FOS (Glisic 2009)

	Extrinsic interferometry (EFPI)	Intrinsic interferometric (SOFO)	Fiber Bragg gratings (FBGs)	Intensity-based (microbending)
Gauge length	50 to 70 mm (2 to 2.75 in.)	250 mm to 20 m (0.82 to 65.6 ft)	10 mm to 2 m (0.4 in. to 6.6 ft)	100 mm to 10 m (4 in. to 32.8 ft)
Multiplexing	Parallel	Parallel	In-line and parallel	Parallel
Maximum number of channels	32	Static: unlimited Dynamic: 8	16 channels 5 to 10 sensors/channel	4/slave, 5 sl./master 4 masters
Stability	Long-term	Static: long-term	Long-term	Long-t. drift 2%
Resolution	$\pm 0.01\%$ full scale	Static: $2\ \mu\text{m}$ (8×10^{-5} in.) Dyn.: $10\ \text{nm}$ (4×10^{-7} in.)	$0.2\ \mu\text{e}$	1 mm (0.04 in)
Repeatability (precision)	Not available	Static: $<0.2\%$ of measured value	$<1\ \mu\text{e}$	1%
Dynamic range	$\pm 3000\ \mu\text{e}$	Stat.: -5000 to $+10,000\ \mu\text{e}$ Dynamic: $\pm 5\ \mu\text{m}$ ($\pm 2 \times 10^{-4}$ in.)	-5000 to $+7500\ \mu\text{e}$ packaging dependent	$5000\ \mu\text{e}$
Temperature influence	Insensitive or packaging dependent	Self-compensated	Compensation needed	$0.6\ \mu\text{m/K}$ (1.3×10^{-8} in/F)
Maximum measured frequency	20 Hz	Static: 0.1 Hz Dynamic: 10 kHz	0.5 MHz	100 Hz

Table 4.5.1.2b—Best performance of distributed strain FOS (Glisic 2009)

	Brillouin OTDA	Brillouin OTDR	Rayleigh
Spatial resolution*	0.5 to 5 m (1.6 to 16 ft)	1 m (3.3 ft)	1 mm (0.04 in.)
Sampling interval	100 mm (4 in.)	50 mm (2 in.)	0.4 mm (0.002 in.)
Maximum number of channels	2, 16 with channel switch	1, channel switch can be added (no. of channels is not available)	1, channel switch can be added (no. of channels is not available)
Stability	Not available	Not available	Not available
Resolution*	$2\ \mu\text{e}$	$30\ \mu\text{e}$ ("accuracy", 2RMS)	$1\ \mu\text{e}$
Repeatability	Not available	$<0.02\%$	Not available
Instrument range	$\pm 30,000\ \mu\text{e}$	$\pm 10,000\ \mu\text{e}$	$\pm 10,000\ \mu\text{e}$
Maximum length of sensor	250 m to 5 km (820 ft to 3.1 miles) for up to 20 db losses	N/A	70 m (230 ft)
Temperature influence	Compensation needed	Compensation needed	Compensation needed
Measurement speed*	10 s to 15 min	4 to 25 min	10 to 250 Hz

*Parameters are mutually correlated.

commercialized type of FOS), but because it uses short-gauge sensors, it falls beyond the scope of this document. Figure 4.5.1.2b shows classification of FOSs based on their gauge length and functional principles and emphasizes in shaded fields the technologies presented in this document. A detailed presentation of all the above technologies can be found in [Measures \(2001\)](#), while their best performances are presented in Tables 4.5.1.2a and 4.5.1.2b.

4.5.1.3 FOS based on Michelson and Mach Zehnder interferometry—Michelson and Mach Zehnder interferometers are similar in appearance. In both cases, light is sent down two optical fibers and then a relative phase shift between these beams is measured in order to measure the difference in length of the two fibers. In the Michelson interferometer, chemical mirrors, silvered at the end of two optical fibers, reflect the light back to the photo detector (refer to Fig. 4.5.1.3a). Originally, a Mach Zehnder interferometer had two outputs instead of mirrors. The light would pass

through the two optical fibers and the difference in their phases would be inferred by a photo detector. However, for practical purposes, a Mach Zehnder interferometer can be modified by adding mirrors at the end of the optical fibers so that the sensor has the same physical appearance as the Michelson interferometer. The difference between the two techniques is in the opto-electronics of the reading unit. A Michelson interferometer-based reading unit allows for very stable, long-term static measurements, but it cannot measure dynamically. On the other hand, a Mach-Zehnder interferometer-based reading unit allows for high-resolution, short-term dynamic measurements.

Long-gauge interferometric sensors have been successfully applied in many projects ([Glisic et al. 2010](#)), and they are known under abbreviation SOFO ([Inaudi 1997](#)). The standard SOFO sensor is composed of active and passive zones. The active zone measures the deformations, while the passive zone serves to carry the information between the

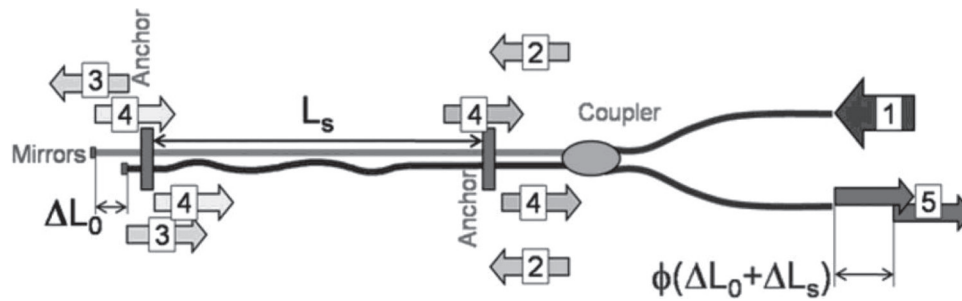


Fig. 4.5.1.3a—Schematic representation of an interferometric SOFO sensor (based on Michelson interferometry). Measurement steps: [1] light is injected into sensor; [2] coupler splits light into two optical fibers; [3] light in reference fiber reflects off the mirror and travels back; [4] light in measurement fiber reflects off the mirror, travels back, and combines in coupler with the light from reference fiber; and [5] combined light signals are decoded to find shift in phase and infer change in strain in sensor (Glisic 2009).

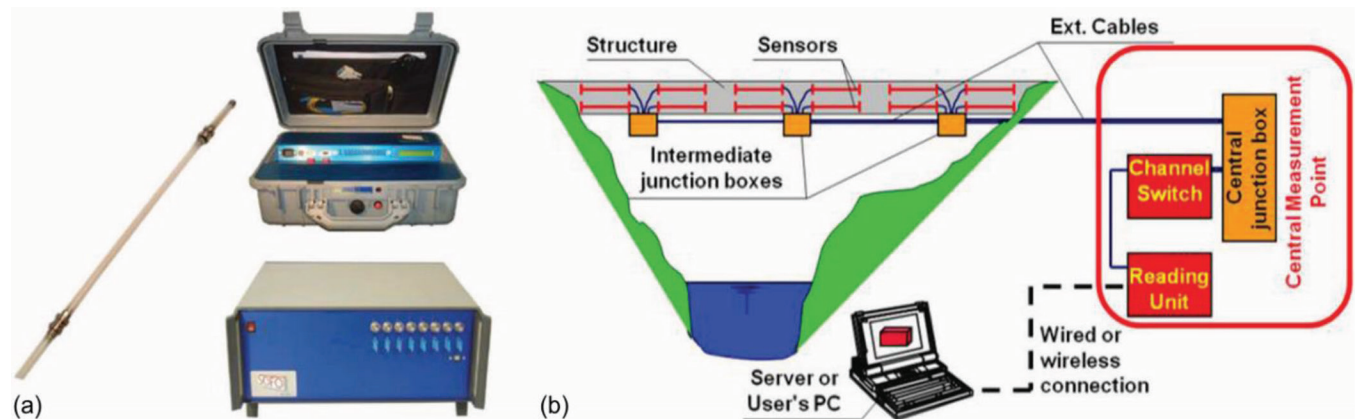


Fig. 4.5.1.3b—(a) Examples of commercially available interferometric SOFO sensor; portable (above) and rack (below) reading units; and (b) a typical application of the SOFO system (courtesy of RocTest/SMARTeC [Glisic 2009]).

active zone and the reading unit. The sensor is schematically represented in Fig. 4.5.1.3a.

The active zone contains two optical fibers placed in a protective tube (not shown in Fig. 4.5.1.3a)—the measurement fiber and the reference fiber—and it is limited by two anchor pieces that serve to attach the sensor to the monitored structure and to transmit the deformation from the structure to the measurement fiber. The measurement fiber is pretensioned between the anchor pieces to measure both the shortening and elongation of the structure. The reference fiber is loose and independent of both the measurement fiber and the deformation of the structure, its purpose being to compensate the measurement fiber for temperature changes. Both fibers have mirrors silvered at their extremities. The passive zone transmits the signal (light) from the reading unit to the sensor and back. It is composed of one single-mode fiber, connector, and coupler, all protected by a plastic tube (two fibers are shown in the Fig. 4.5.1.3a to simplify presentation; however, only one fiber is used in practical applications).

The reading unit sends the light down the fiber in the passive zone [1], the coupler splits it and guides it into two fibers of active zone [2], where it travels to the extremities and reflects off the mirrors, first from the shorter reference fiber [3] and then from the longer measurement fiber [4]. Because the two fibers have different lengths, a shift in phase is created between the two reflected lights, and this shift in

phase is proportional to the length difference in the optical fibers. The change in deformation of the monitored structure affects only the length (that is, strain) of the measurement fiber while the reference fiber remains unchanged. Consequently, the shift in phase will change. The signals from the two fibers are combined [5] and transmitted back to the reading unit, where they are analyzed and converted into strain. The SOFO sensor is considered a true long-gauge sensor because the light integrates the strain along the measurement fiber between the anchors (that is, along the gauge length). Examples of SOFO sensors as well as portable and rack reading units are given in Fig. 4.5.1.3b.

The static reading unit, based on Michelson interferometer, can be complemented with a channel switch to enable the reading of multiple sensors. A multi-fiber cable extension connects the switch and sensors. A typical application of the SOFO system is shown in Fig. 4.5.1.3b(b). The performance specifications of the SOFO system are given in Table 4.5.1.2a. For more information, refer to Inaudi (1997).

4.5.1.4 FOS based on fiber Bragg gratings (FBGs)—Fiber Bragg gratings (FBGs) consist of periodical changes of the refractive index created in the fiber core by appropriate exposure to ultraviolet light (Dadpay et al. 2008). An FBG behaves as a partial mirror: if light containing a certain range of wavelengths travels through the fiber with an FBG, the FBG will reflect one specific wavelength, frequently called

the reference wavelength, while letting all the other wavelengths pass through, as shown in Fig. 4.5.1.4a. The magnitude of the reference wavelength depends on the optical properties of the FBG, which is defined during the manufacturing process. The refractive index of silica and its geometrical properties depend linearly on strain and temperature. Thus, an increase in one or both of these parameters will change the magnitude of the reflected wavelength. Consequently, the change in strain or temperature can be determined from the change in magnitude of reflected wavelength by a linear conversion equation. It is important to highlight that the change in magnitude of the wavelength depends on both strain and temperature simultaneously and, therefore, strain sensors must be compensated for temperature using an appropriate compensation procedure (commonly, temperature is measured separately by an FBG temperature sensor and used for compensation purposes).

A typical physical length of an FBG is approximately 5 to 13 mm (0.2 to 0.5 in.) and, depending on packaging, FBG sensors can be used either as short-gauge sensors (the fiber-optic equivalent of a classical strain gauge) or as long-gauge sensors (the fiber with a FBG is pretensioned between two anchoring points similar to interferometric SOFO sensors). There is a large variety of both types of the sensor available on the market, and examples are given in Fig. 4.5.1.4b.

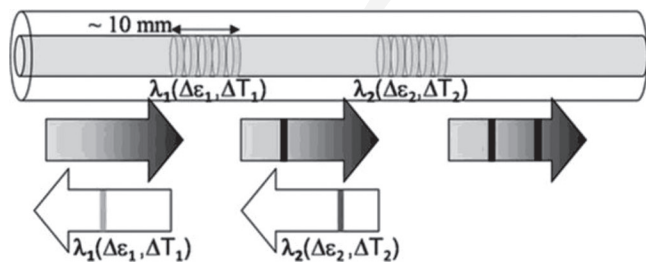


Fig. 4.5.1.4a—Schematic representation of an FBG sensor with two FBGs inscribed along the same optical fiber and having different reference wavelengths λ_1 and λ_2 (Glisic 2009).

The reading unit for FBG sensors contains both the light source and decoder. The light source sends a spectrum of wavelengths toward the FBGs in the fiber, while the decoder measures the intensity of the back-reflected light and determines the magnitude of the reflected wavelength using a tunable laser with a wavelength filter (for example, Fabry-Perot cavity) or spectrometer. Examples of reading units are given in Fig. 4.5.1.4b.

It is important to note that while one specific wavelength is reflected by an FBG, all the other wavelengths contained in the spectrum actually pass through. This allows for several FBGs to be inscribed and read along the same fiber as long as their reference wavelengths are all mutually exclusive (in-line multiplexing). This represents a significant advantage of FBGs over other strain-sensing technologies, as several sensors can be read from a single channel of the reading unit. The maximum number of sensors that can be placed along a single fiber depends on the expected magnitude of strain and temperature variations in the monitored structure, and for civil engineering applications, it ranges between five and 10 sensors per fiber. An FBG reading unit has one to four channels, and an additional channel switch can be added for a maximum of 16 channels. Taking into account that each channel can read five to 10 in-line (serially) connected sensors, the maximum number of sensors that can be served by a single reading unit ranges between 80 and 160. A typical connection scheme for FBG systems is similar to the one shown in Fig. 4.5.1.3b(b), with the addition that groups of neighboring sensors could be mutually (inline) interconnected. The performance specifications of commercially available FBG systems are given in Table 4.5.1.2a.

4.5.1.5 Distributed FOS based on Rayleigh and Brillouin scattering—Distributed sensing enabled by fiber-optic technology has resulted in paradigm shift in many engineering fields that deal with sensing and monitoring. Distributed sensors (or sensing cables) can be represented by a single cable, which is sensitive at every point along its length (Glisic and Inaudi 2007). Hence, one distributed sensor can



Fig. 4.5.1.4b—Examples of commercially available FBG sensors (both short- and long-gauge) and reading units (courtesy of Roctest/SMARTeC, Micron Optics, and Fiber Sensing) (Glisic 2009).

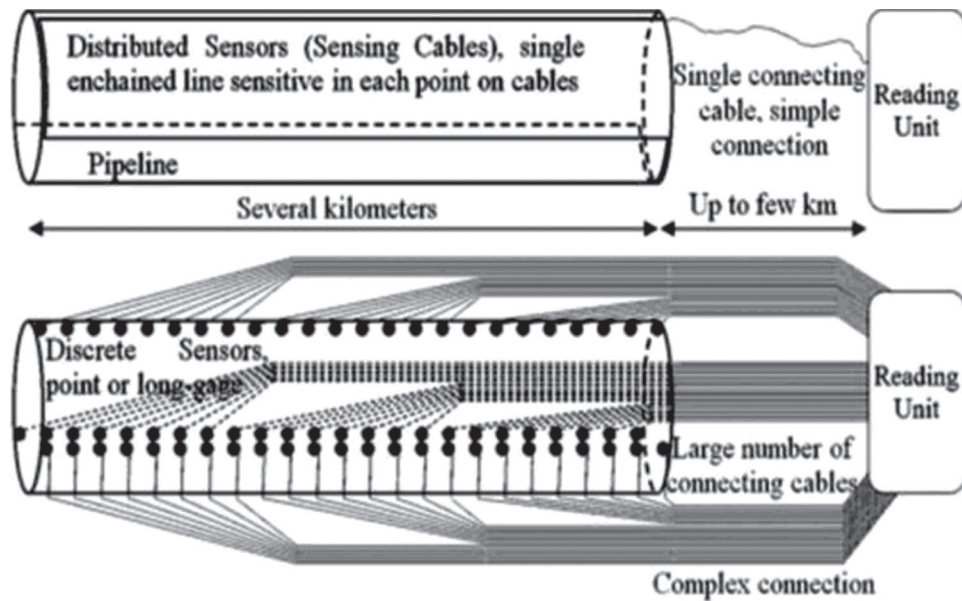


Fig. 4.5.1.5a—Distributed versus discrete monitoring: schematic comparison (Glisic and Yao 2012). (Note: 1 km = 0.62 miles.)

replace thousands of discrete sensors (long- or short-gauge). The connecting scheme for distributed sensing is different from that used for discrete sensors: one distributed sensor requires a single connection cable to transmit the information from all the measuring points to the reading unit as opposed to a number of connecting cables required in case of wired discrete sensors. In addition, distributed sensors might be easier and more economical to install and operate in the case of large structures (which does not necessarily apply in the case of small structures). An illustrative comparison between distributed and discrete sensing is shown in Fig. 4.5.1.5a (the example is given for pipelines but similar schemes apply for other types of structures).

Although a distributed sensor is sensitive to the measurand (strain and/or temperature) at every point along its length, it provides the user with measurements at discrete points that are a constant distance apart, called the sampling interval, and the value of the measurand is actually an average measured over a certain length, called the spatial resolution (Lanticq et al. 2009). Practically, the spatial resolution represents the gauge length of the measurement. The sampling interval and the spatial resolution are two configuration parameters, characteristic of distributed systems, and they can be set by the user, depending on the project requirements. However, these two parameters are correlated with the resolution and precision of the measurement as well as with the measuring time. Consequently, trade-offs must be made to optimize all these parameters, depending on the application. For example, very precise and localized measurements (small spatial resolution) are slow (tens of minutes in the case of Brillouin-based system), while increasing the sampling speed (few minutes) affects one or both of the aforementioned parameters.

The three main physical principles that are measured for distributed sensing are: 1) Rayleigh scattering (Posey et al. 2000); 2) Brillouin scattering (Kurashima et al. 1990); and 3) Raman scattering (Kikuchi et al. 1988). Each principle results in its own measurement technique that is based on

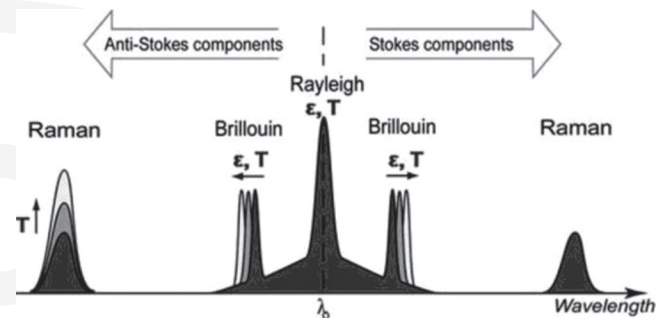


Fig. 4.5.1.5b—Scattered light properties as encoding parameter for strain and/or temperature measurements (modified from Glisic and Inaudi [2007], courtesy of SMARTEC SA).

the relation between the measurand (strain and/or temperature) and the change in optical properties of the scattered light, as shown schematically in Fig. 4.5.1.5b. Raman scattering is sensitive to temperature only, and that is why it is not presented in more detail in this document. Rayleigh scattering is based on the shifts in the local Rayleigh backscatter pattern, which is dependent on the strain and the temperature, and, consequently, the strain measurements must be compensated for temperature. The main advantages of Rayleigh-based systems are the short measurement time (dynamic measurements for limited sensor lengths), high resolution of measured parameters, and short spatial resolution, but the challenge is in limited maximum length of sensor of only 70 m (230 ft) (Lanticq et al. 2009). Thus, this system is suitable for monitoring of localized strain changes over relatively short distances. Performance specifications for the Rayleigh system are given in Table 4.5.1.2b.

Brillouin scattering is based on the change in frequency of Brillouin scattered light, which is dependent on the strain and the temperature. Consequently, the strain measurements must be compensated for temperature. The strain measurements for the FBG and Rayleigh-based FOS. Thermal compen-



Fig. 4.5.1.5c—Example of commercially available BOTDA reading unit and sensors (courtesy of Roctest/SMARTec and Omnisens) (Glastic 2009).

sation measurements are frequently performed with cables containing a loose (non-bonded) optical fiber. Two sensing techniques have been developed based on Brillouin scattering: spontaneous Brillouin optical time domain reflectometry (BOTDR) (Wait and Hartog 2001), and stimulated Brillouin optical time domain analysis (BOTDA) (Nikles et al. 1996, 1997). BOTDA requires access to both ends of the cable whereas BOTDR only requires access from one end, which makes the latter more robust if the cable is in risk of damage. BOTDA is, in general, less sensitive to cumulative optical losses that may be generated in the sensing cable due to manufacturing and installation, and allows for monitoring of exceptionally large lengths (Thevenaz et al. 1999). For example, depending on the packaging of the distributed sensor, a single reading unit with two channels can take strain measurements over lengths of up to 10 km (6.2 miles) (each channel can operate up to 5 km [3.1 miles]). Remote modules can be used to triple the monitoring lengths, and channel switching of up to 16 channels can be added as well. An example of a commercially available BOTDA reading unit with sensors is given in Fig. 4.5.1.5c.

The measurement specifications in terms of strain resolution and measuring time for Brillouin-based systems are not as good as those of discrete sensors and the Rayleigh-based systems; however, the important advantage of Brillouin-based systems is the significantly longer length of the sensor (several kilometers [miles]) and, consequently, greater monitoring coverage of the structure. Hence, the Brillouin-based systems are better suited for monitoring global strain changes over large areas of structures. The performance specifications for strain monitoring achievable with Brillouin-based systems are given in Table 4.5.1.2b.

4.5.2 Equipment needed—The equipment needed to perform strain measurement using FOS is given in the following sections.

4.5.2.1 Sensors—All types of FOS strain sensors are commercially available, and examples of sensors are shown in Fig. 4.5.1.3b, 4.5.1.4b, and 4.5.1.5c.

4.5.2.2 Installation equipment and material (adhesives, brackets, tools)—Depending on their packaging, the FOS strain sensors can be embedded in concrete or surface-mounted by means of brackets or gluing. In the case of embedding, the sensors can be attached to the reinforcing bars or placed between the reinforcing bars using plastic ties or ordinary wire. For surface mounting, special brackets

can be used (usually provided by the manufacturer of the sensors). The brackets can be attached to the structure by welding, bolting, fastening, or gluing, depending on the type of material and the allowed procedure; thus, the associated tools and material (for example, welding device, drill, bolts, and nuts) should be provided. In the case of gluing, the type of adhesive depends on the material of the monitored structure (typically concrete, steel, or composite material), and it is usually recommended (on request) and delivered by the sensor manufacturer. To achieve correct strain transfer from the structure to the strain sensor and to ensure durability of installation, the surface of the structure must be smoothed and well-cleaned before the gluing. Regardless of the type of installation, the surface-mounted sensors should be protected from the elements (snow, rain, and wind), rodents, and mechanical damage by external enclosures, which are installed so that they do not interfere in any manner with the sensors.

4.5.2.3 Cable extensions, connections, and junction boxes—The cable extensions connect sensors with the reading unit, and they are usually provided by the manufacturer of sensors. To minimize the costs, cables that are protected from infiltration of water, rodents, and mechanical damage (for example, windborne debris, unintentional cutting, and vandalism) are recommended. Otherwise, additional protection might be needed, leading to extra costs. Connections between optical fibers are made either by pre-installed connectors or by splicing. Splicing requires expensive equipment (splicing machine, cleaver, fiber-optic stripper) and trained personnel, which might be difficult and expensive to provide on-site. Thus, rather than splicing, the use of mechanical optical fiber connectors for the connections is highly recommended, as they are simple and easy to perform. There is a large variety of connectors available and the most popular are FC-PC, FC-APC (angle 8 degrees), E2000, and E2000-APC (angle 8 degrees). Angled connectors (APC), in general, have lower losses than non-angled connectors. Optical connectors are all either male or female and, thus, every connection requires a mating adapter, which is either double female or double male, respectively. Hybrid mating adapters are available, so different types of connectors can be mated as long as they are both either angled or non-angled. An angled connector cannot be connected to a non-angled connector due to the gap in angle. To help with identification of connectors, angled connectors are usually colored green. While connecting optical fibers, it is important to keep the connectors perfectly clean, and a cleaning kit should be provided (it is usually sufficient to have widely available products such as isopropyl alcohol and lens-cleaning paper sheets). Junction boxes host and protect the connections between various cables. Consequently, the junction boxes should be dust-proof and waterproof, and robust enough to prevent damage to the connections.

Discrete FOS are frequently delivered with a lead cable of limited length and preinstalled connectors. Hence, the lead cables from groups of neighboring sensors can be guided to a single point (junction point) where all the cables can be extended by multifiber cable, and its extension guided to

the reading unit. The connections should be protected by a junction box.

Distributed FOS can be delivered with or without connectors. In the latter case, the connectors should be spliced to the cable, and it is recommended to perform this operation in a protected area, because the splicing involves sensitive and expensive equipment. The range of the distributed FOS system strongly depends on the quality of the connection (lower connection losses imply longer sensing ranges) and, thus, it is recommended to pay attention to the quality of the connections and minimize the total number of intermittent connections (by splices or by connectors).

4.5.2.4 Reading units (data acquisition hardware)—Numerous types of reading units are commercially available for all types of FOS. Commonly, they all need a regular electrical power source. They can also be provided with autonomous recording and remote access via a local area network, internet, or mobile communication network. If the reading unit is installed on-site, it is recommended to protect it with a dust-proof and waterproof rack equipped with an air-conditioning system that will maintain the temperature and humidity within the functioning range of the reading unit.

4.5.2.5 Computers and external hard drives—A standard personal computer (PC) is frequently needed to set the data acquisition parameters, store the data, and host the data management and analysis software. In some cases, the computer is a part of (integrated in) the reading unit. External drives can be used for backup and data archival.

4.5.2.6 Data management and analysis software—Data management software enables access, visualization, and storage of data. It is usually provided by the manufacturer of the reading unit. In many cases, data processing and basic setting of thresholds are included. Data analysis software enables transformation of data into information on structural condition. At the moment, the data analysis software is custom-made depending on the application, and frequently has limited capabilities.

4.5.3 Method of deployment—While planning to monitor the strain, it is important to understand that the sources of strain in construction materials are manifold (Muravljov 1989). Strain sensors cannot distinguish between various strain components; they always measure total strain—that is, the sum of all the components. Consequently, if separation of the strain components is needed (and this is frequently the case, as the most interesting component to measure and monitor is usually mechanical strain), measurement procedures and data analysis algorithms that enable this separation have to be established (ideally before the sensors are applied to the structure). More information regarding the strain components can be found in Section 4.1.8. Specific methods for deployment of FOSs are practically summarized in Sections 4.5.2.2 to 4.5.2.6; also refer to Fig. 4.5.1.3b and 4.5.1.5a.

4.5.4 Strengths and limitations

4.5.4.1 General strengths and limitations—Research on the use of standard telecommunication optical fibers for sensing purposes in civil engineering has flourished during the last two decades. Thus, FOSs have reached the market maturity stage, and their acknowledged strengths are their

applicability in harsh environments, high precision, excellent long-term stability, and exceptional durability against many deterioration processes that affect other sensors (Udd 2006; Glisic and Inaudi 2007). The silica is chemically stable and is a relatively inert material, while the light that is used for sensing purposes is not affected by electromagnetic interference frequently found on-site (for example, due to the close proximity of power lines or lightning). These are contributing factors to the long-term stability and reliability of FOS networks. In addition, the ability to measure over distances of kilometers (miles) with no need for electrically active components is an important strength when monitoring large and remote structures, such as landmark bridges, dams, tunnels, and pipelines (Udd 2006). The most significant practical limitation with FOSs relates to the repair of cables, which requires on-site splicing that involves expensive equipment and trained personnel. Another limitation is the general perception that FOS technologies are expensive. While the costs of basic FOS components (sensors, reading unit, and cable extension) are higher than those of electrical strain sensors, the long-term stability and lower installation costs of FOS (especially compared with the increased costs associated with shielding of electrical cables and sensors) make them competitive for long-term monitoring.

Tables 4.5.1.2a and 4.5.1.2b compare strain FOSs and can be used to aid selection of an FOS system for a specific application. EFPI-based and intensity-based sensors, although out of the scope of this section, are also presented in Table 4.5.1.2a for the purposes of comparison and completeness. The tables were compiled based on information available on datasheets of commercially available products provided by their manufacturers. It is important to highlight that the tables contain the best achievable performance of each technique, while the final on-site performance may depend on the specific application (for example, installation method and specific sensor packaging).

4.5.4.2 Strengths and limitations of interferometric sensors (Michelson and Mach Zehnder)—Besides the general strengths of FOSs mentioned in the previous section, additional strengths of interferometric sensors (SOFOs) are their thermal self-compensation (practically insensitive to temperature), extremely high resolution ($2\text{ }\mu\text{m}$ [$8 \times 10^{-5}\text{ in.}$] over any gauge length), and unlimited number of channels for static measurements. A major limitation is the difficulty in combining static and dynamic measurements because the reading units for these two purposes are different. Also, while the number of channels (that is, the number of sensors) served by a single static reading unit is practically unlimited, the number of channels for the dynamic reading unit, based on Mach Zehnder interferometry, is limited to eight.

4.5.4.3 Strengths and limitations of FBG sensors—The principal strength of FBG sensors is in-line multiplexing (quasi-distributed sensing). An additional advantage of FBG sensors is the capability to perform both static and dynamic measurements with the same reading unit. A limitation is thermal compensation that requires an additional temperature compensation FBG for each strain measurement point.