

Serviceability and Safety: Core Aspects of Sustainable Structures

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Synopsis: Serviceability, safety, and resilience are core components of a sustainable structure. This paper will discuss the key issues related to economic, social, and environmental factors. Structural efficiency plays an important role in reducing material usage while maintaining the necessary serviceability (including durability) of a structure. Resilience against both manmade events (such as terrorist attacks) and forces of nature (such as tsunamis, hurricanes, tornados, and flooding) has come to the forefront in recent years since longevity of a structure is central to sustainability. Wrapped into each of these decisions related to serviceability and safety/resilience are the potential environmental concerns related to choice of materials. Each of these topics is discussed within the context of concrete structures.

Keywords: sustainability, green, resilience, safety, environment

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INTRODUCTION

Sustainability Defined

While there are many interpretations of the meaning of the term “sustainability,” the most commonly accepted definition is from the Bruntland Commission report (1987): “Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.” This framework is not limited to only green or environmental aspects. Sustainable designs include economic, social, and environmental considerations.

SUSTAINABLE ASPECTS

Environmental

Environmental issues related to sustainable building design and construction can be grouped in various ways. Fig. 1 illustrates a grouping that aligns with LEED (Leadership in Energy and Environmental Design) for building construction.



Fig. 1—One representation of the environmentally related components for buildings (Schokker, 2010).

Materials—The use of perceived green materials is one of the most common examples cited in sustainability discussions. Materials such as bamboo and cork are touted for their rapid growth that makes their use potentially sustainable. The most sustainable material is one that is never used at all (by reducing material needs). In concrete, material reduction can be achieved by structural efficiency through use of higher strength concrete, prestressing, and many other design choices. Supplementary

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cementitious materials also can reduce the use of natural resources necessary to make cement. If material reduction is not possible or is limited, the next best alternative is to reuse materials. When a structure or structural components can be repaired or retrofitted, this is typically a far more sustainable option than demolition followed by new construction. With the large building inventory already in place, retrofit/repair is growing and is expected to continue to increase (Bureau of Labor Statistics, 2010). Components such as panels, pavers, block, and other materials can also be removed intact from one site and reused on another with few changes made to the component itself.

Recycling typically is mentioned when environmental concerns are discussed. Reduction and reuse are preferable to recycling, but recycling can have a major contribution to sustainability. Recycling materials not only keeps them from going into landfills, but also can reduce the amount of new material needed. During demolition of a concrete structure, the steel reinforcement can be recycled as well as the concrete itself. The concrete can be crushed to form recycled concrete aggregate for use in new concrete. ACI Committee 555 has a report that provides details on the use of recycled aggregate (2001). Waste products such as tire, glass, fibers, and many others have also found a home in some concrete applications. Waste packaging from the construction site should be recycled (and ideally, packaging is minimized in the first place).

Site Selection—Site selection considerations include location to services and preservation of green space. Clean-up of Brownfield sites for use as sites for new construction is particularly encouraged over use of new land. Building footprints and traditional paving with concrete or asphalt should be minimized. The effects of paving can be reduced through use of permeable concrete or interlocking permeable concrete pavers.

Water—Water is a resource that continues to be strained more each year throughout the world. As part of the building industry we can contribute in two major areas: stormwater management and reduction of water use. Permeable concrete and paver systems provide access for runoff to get back to the aquifers directly while filtering out debris. These systems can provide a holding area for water during heavy rain events that allows the water to slowly filtrate into the soil. Concrete cisterns can be used to collect and store water for other uses on-site. Green roofs can provide multiple benefits including additional green space, stormwater filtration and quantity management, and a reduction in the heat island effect.

Air Quality—Ventilation, mold control, and the use of low emitting (low volatile organic [VOC] compound) materials is critical for maintaining good air quality in buildings. Carpet, paint, sealants, stains and other finishes must be carefully controlled to maintain air quality. Concrete adapts well as a bare finish and aesthetics can be enhanced through use of pigments, form liners and stamping. Concrete also resists mold growth.

Energy—Energy use can be divided into embodied energy and operational energy. Embodied energy includes all of the energy used in the materials and construction of the physical building. The manufacture and transport of materials is a part of the embodied energy. Operational energy is the energy needed to heat, cool, light, and operate a building. The operational energy is typically significantly larger than the embodied energy and this gap increases as the service life of the building increases. However, as we move toward use of renewable energy and more efficient systems for heating/cooling and lighting, the operational energy decreases.

Concrete contributes significantly to the embodied energy of a building in a large part due to the manufacture of cement. The cement contribution to energy use can be reduced through more efficient kilns, use of waste products or by-products for fuel, or substitution of a percentage of the cement with supplementary cementitious materials in the final concrete mixture.

When considering the overall energy impact of a structure, the area where concrete stands out is in its ability to reduce the operational energy of a building. Concrete has the advantage of high thermal mass that can effectively reduce energy needs by lowering peak energy needs and shifting the time of maximum demand (thus moving demand to off peak times). Fig. 2 illustrates this effect. When combined with an effective insulating layer (such as in precast sandwich panels) with little thermal bridging, concrete can provide the tight envelope that is crucial for significantly lowering energy demand.

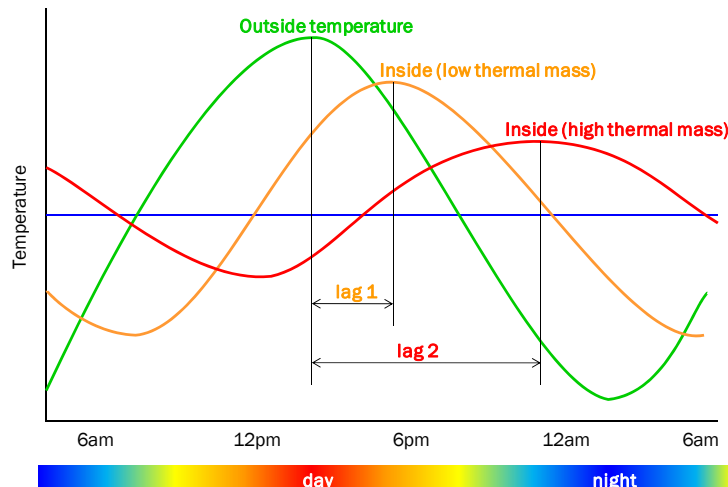


Fig. 2—The effect of thermal mass on moderating and shifting interior temperatures (Schokker, 2010).

Social

The social aspects surrounding a building's design, construction, and use form one of the three pillars of sustainability. This includes humanitarian issues as well as workplace related human factors such as building acoustics, aesthetics, and daylighting. The heat island effect is also included in this category for this discussion, but (like other topics) could also be in one of the other categories.

Community issues such as use of the local workforce or influence on local industry are integral to a building's sustainability. A building that doesn't become a part of the community (socially, economically, or otherwise) has little chance of lasting long-term. Our building infrastructure needs must be focused on service lives measured in centuries, not decades. The specifics of durability, safety and serviceability are covered in detail in later sections.

Studies in human factors combine the aspects of the workplace, people, and management. A building must provide a workplace that allows occupants to be productive. This includes comfort in temperature, day lighting, sound, and aesthetics. Temperature regulating effects were discussed in the previous section related to thermal mass. Effective day lighting for the work environment can be provided through the combination of exterior windows, selective shading from direct sunlight in work areas, and use of light colored finishes (such as concrete or light colored painted surfaces). For acoustic considerations, concrete is effective in reflecting sound, making it a good choice for reducing sound transfer between rooms. Furnishings and people contribute to the sound absorption that is needed within a room than concrete does not provide. Aesthetics can often be considered "a matter of taste," but concrete has long established itself as a choice that combines flexibility of shape, color, and texture that is popular with architects, engineers, and the public.

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Economic

The economic pillar of sustainability is focused on both direct and indirect monetary costs. In a building structure, this includes the upfront cost as well as life-cycle costs. The concrete industry has long worked toward providing economic options for structures. Since durability is forefront in the benefits of concrete structures, life-cycle cost is particularly important to understanding the overall building cost. The energy benefits discussed in previous sections combined with the length of service life provided by a durable structure are the biggest impacts on reducing overall costs.

SAFETY AND SERVICEABILITY

Functional Resilience

“Functional resilience” has developed as the term in the sustainable building industry that encompasses the safety, durability, disaster resistance, adaptability, and other aspects that relate to creating a structure that sustains the unpredictable environment that it will see over potentially a century or more of use. A building with renewable energy, reused or recycled materials, wonderful day lighting and air quality that is demolished by a hurricane after a few years of service is not a sustainable building. Functional resilience must be fully integrated into sustainable buildings and concrete has a strong record in this area.

Fire—Concrete is inherently fire resistant and has no additional requirements for fire protection for most applications, so material and money can be saved. Concrete walls and other members provide not only fire resistance, but also can play a crucial role in fire containment and in keeping routes of egress clear. This provides a substantial safety increase for occupants.

Wind and Water—Heavy rainfall events, wind storms, hurricanes and tornados can leave structures devastated. Concrete structures perform well under these conditions due to their ability to resist high winds. The concrete can also remain uncompromised even after submersion in water for extended periods of time. While mold and rot are a concern with many materials, concrete resists both and thus not only positions a structure to go quickly back into service, but also reduces health concerns related to the removal of molded or rotted materials. Fig. 3 shows a concrete house after hurricane Katrina. The high water mark on the house was 28 feet (8.5 m) above the base of the structure and the house was the only one left standing in the neighborhood. Fig. 4 shows a board impaled in a concrete wall after a tornado in Georgia. Concrete walls can be very effective in protecting occupants from flying debris.



Fig. 3—This concrete house (under construction during hurricane Katrina) was the only house left standing in the neighborhood (photo courtesy of John Fleck, FEMA)

Earthquakes—Earthquakes can produce extreme loads on a structure, and specialized building design procedures are used to achieve a building that can withstand the forces. Seismic design includes provisions for ductile performance and separation of participating versus non-participating structural elements. Various materials can be used for building systems that will minimize structural damage and protect occupants. Many types of concrete systems can be used and innovations continue in this area to achieve more ductile and robust structures. Earthquakes can also trigger fires and tsunamis, so concrete's ability to resist water and fire damage also factors in to the safety for occupants during earthquakes.

Blast and Impact—Design for blast and impact loads has become more prevalent in the past decade as concerns rise about potential terrorist attacks. Mass concrete is a popular choice for barriers to resist blast and impact. Properly detailed concrete elements can effectively resist blast load damage as well as reduce the potential for shrapnel from failed portions of the structure. The “United Facilities Criteria: Structures to Resist the Effects of Accidental Explosions” gives details on blast resistant design (U.S. Department of Defense, 2008). Concrete spalls on the interior side of walls can also be resisted through the use of innovative fibers particularly for this purpose (Coughlin et al, 2010).



Fig. 4—During a tornado in Georgia, this awning was ripped apart and board was impaled into an adjacent concrete wall (inset) (photo courtesy of Mike Moore, FEMA)

Pests—Pests such as termites can heavily damage wood structures in short periods of time. Damage to a wood structure's framing can occur without outward visual indications if siding or other cladding covers the wood. Termites and other pests cannot feed on concrete.

Durability—Long-term durability and corrosion resistance is critical to serviceability. Concrete has a long track record of proven durability in structures dating from ancient times. It also provides a high pH environment to protect the steel reinforcement used in structural concrete. Proper design (mix design, structural design, detailing) and construction (placement, quality control, curing, etc) are the key to getting a durable concrete structure.

Building Codes

Building codes are based on providing health and safety to occupants. Green rating (certification) systems such as LEED (Leadership in Energy and Environmental Design) have provided the basis for increased awareness of the design, construction, maintenance and operations of sustainable buildings. However, these rating/certification systems are not codes and are not written in mandatory language. They also have not adequately address the functional resilience aspect that is integral to true sustainability.

Green codes are being developed at levels from local jurisdictions up to international levels and are beginning to recognize the importance of functional resilience. The American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) has published Standard 189.1, *Standard for the Design of High-Performance Green Buildings except low-Rise Residential Buildings* (2009). The *International Green Construction Code* (IGCC) is currently under development with the second public release out for comment. The code is anticipated to be published in early 2012.

SUMMARY

As the concrete industry moves toward designing, constructing, and maintaining more sustainable structures, we must incorporate functional resilience at the core. Safety and serviceability are fundamental for any structure. When spending considerable effort and money for sustainable buildings, the safety and serviceability must also take a step up to account for extreme events and previously unforeseen catastrophes.

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Impact of Adjacent Lane Traffic on the Serviceability of Concrete Bridge Decks

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Synopsis: On major highways, bridge deck replacement often involves a diversion of traffic by shifting the travel lane adjacent to the repaired roadway. Thus, any traffic vibration or differential deflection, induced transversely due to truck traffic in the adjacent lanes, can affect the fresh concrete. Although there is very little evidence that traffic vibration in the adjacent lanes has affected the serviceability of concrete bridges in the past, more recently there have been some concerns about pouring high-performance concrete (HPC) in adjacent lanes because transverse cracking has been observed on newly repaired bridge decks. This paper examines the impact of traffic diversions and deflections that are induced transversely by truck traffic in adjacent lanes on the serviceability of bridge decks. In addition to field monitoring and laboratory testing, finite element analyses were used to simulate various bridge deck systems, traffic patterns, and truck loads to determine their effects on the serviceability of bridge decks. Results indicated that traffic loads and patterns can adversely affect the serviceability of concrete bridge decks. The results revealed that specific modifications to construction procedures and materials can significantly reduce the degree of transverse cracking in bridge decks.

Keywords: Bridge Deck; Finite Element Analysis; Traffic Vibration; Cracking; Serviceability.

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INTRODUCTION

Concrete durability is one of the major concerns for many State Departments of Transportation (DOT), as well as other Transportation Authorities and the Federal Highway Administration (FHWA). One method of improving the durability and performance of concrete is to add pozzolans (such as silica fume, fly ash, and slag) to concrete mixtures. This led to the development of high-performance concrete (HPC), which is being currently implemented and used by many States in transportation structures with an emphasis on bridge decks. However, HPC requires applying proper placing and curing practices in order to minimize shrinkage effects. With the frequent use of HPC in applications such as bridge decks, many questions were raised regarding its long-term performance as well as early-age behavior. Moreover, many resources are allocated for implementing and producing HPC for its enhanced impermeability performance against chloride attacks and resistance to abrasion. However, with early-age cracking, the enhanced durability aspects start diminishing with time. Therefore, there is a need to understand the early age behavior of HPC under various adverse loading and placing conditions as well as its shrinkage behavior and potential for cracking under restrained conditions.

The effect of vibration (e.g., due to blasting, jarring, pile driving, and moving traffic) on fresh concrete has been a topic of concern for many years. One of the earliest research efforts to study the impact of vibration on fresh concrete was conducted by Abram in 1919. He performed a laboratory experiment to simulate various vibration frequencies on fresh concrete and their impact on the concrete strength, which proved to have little detrimental effect on concrete. It took another five decades for another research study (Whiffen and Leonard, 1971) to be conducted on the effect of vibration, but this time mainly focusing on bridges, particularly bridge deck widening. Additional, similar research studies followed (Furr and Fouad, 1982; Harsh and Darwin, 1983, 1986; Manning, 1981; Dunham et al., 2007; Issa et al., 2000), including a National Cooperative Highway Research Program (NCHRP) Synthesis (Arnold, 1980) to study the impact of adjacent traffic on bridge widening. Various parameters, including concrete slump, covers, water content, water-to-cement (w/c) ratio, and rebar size, were considered in many of these studies. The compressive, tensile, and bond strengths were the primary properties that were used to evaluate the influence of adjacent traffic, but other measurements, including deflection and visual inspection of bridges and cores were also used. More detailed summaries of these research studies can be found in Issa (1999) and the ACI Committee 345 (2005) report on "Guide for Widening Highway Bridges." Overall, all studies indicated that damage due to traffic-induced vibration is very rare and can be controlled by: i) use of moderate slump of 2 to 3 in. (50 to 75 mm); ii) good reinforcing details; and iii) good forming details. Despite all these research studies, one of the limitations is they do not address high-performance concrete (HPC). Unlike ordinary-performance concrete (OPC), HPC requires various chemical admixtures to alter its placeability and setting time. As a result, some of these effects may lead

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to transverse cracking, especially for bridge deck rehabilitation or widening when adjacent lanes need to remain open to ensure minimal disruption of the traffic. Thus, there is a need to investigate the effects of truck traffic in adjacent lanes, that could induce transverse differential deflection, on bridge-decks cracking behavior.

RESEARCH SIGNIFICANCE

The main objective of this study is to evaluate the cracking behavior of HPC bridge decks and explore the cause of the cracking related to design procedures and impact of adjacent lane traffic. Using a three-dimensional finite element (FE) method as an analysis tool, this study identifies the design procedures and parameters that most directly relate to the severity of cracking in bridge decks. The study provides a new methodology that successfully selects the appropriate design modifications and construction guidelines that minimize the cracking potential of decks for girder bridges.

RESEARCH APPROACH

To study the cracking behavior of bridge decks due to transverse differential deflection induced by truck traffic in adjacent lanes, a detailed three-dimensional FE model was developed. A general-purpose finite element program, ABAQUS, was utilized to generate the model. ABAQUS includes a variety of routines that allow defining specific material behaviors and properties such as concrete cracking and tension stiffening, reinforcing steel rebar, boundary conditions, bond behavior, (e.g., shear studs) and interaction between the steel deck-reinforcement and concrete. The concrete deck was modeled using shell elements, while the beams were modeled using beam elements.

Bridge Model

Two types of bridges were considered in this study: 1) a slab-on-girder bridge and 2) a through-girder bridge. The slab-on-girder bridge consisted of a 72 m (236 ft.) three-span continuous bridge. Each span was 24 m (78.8 ft.) long and 12.5 m (41 ft.) wide. The bridge consisted of six girders spaced at 2.2 m (7.25 ft.) center-to-center to support a 178 mm (7 in.) thick slab. The through-girder bridge was 51.2 m (168 ft.) long and 24.7 m (81 ft.) wide. There were seven floor beams supporting ten 5-span continuous stringers that were spaced at 2.51 m (8.25 ft.) center-to-center. The concrete slab thickness was 165 mm (6.5 in.). Figure 1 illustrates the through-girder FE model.

Various element types were used to model the bridge. The beam element was used to assemble the trusses, floor beams, and stringers. The beam element is a one-dimensional line element that cannot deform in its own plane; under bending the plane sections remain plane. However, ABAQUS includes warping theory—non-uniform out-of-plane deformation of the cross section—for certain beam-elements, particularly open thin-walled beam elements such as an I-beam. Thus, for large plastic deformation, a constraint has to be placed at the support or joint to increase the torsional stiffness. For these bridges, the constraint was ignored because the truck load does not cause large plastic deformation. Hence, standard Timoshenko beams, which allow shear deformation, were used.

Two types of beam elements were chosen for the analysis: (1) two-node linear beams (B31) and (2) three-node quadratic beam (B32) elements. Both beam elements were modeled in spaces with six degrees of freedom at each node. The B31 element has one integration point, whereas the B32 element has two integration points along the length of the beam. ABAQUS also includes an I-beam section in the beam-element cross-section library. The advantage of using the cross-section library was that the moment of inertia and torsional rigidity were automatically calculated.

The shell element was used to model the concrete slab. The reason for choosing a shell element was because the concrete slab had one dimension that was significantly smaller than the others (i.e., the thickness of each slab was smaller than its width and length). ABAQUS contains a vast library of shell elements but the most common and general type of shell element is the four-node shell element (S4). This element is a fully integrated, general purpose, finite-membrane-strain shell element that allows in-plane bending. It also allows transverse shear deformation and uses thick-shell theory; as shell thickness

increases, the Kirchhoff thin-shell thickness decreases. The S4 element has six degrees of freedom at each node and four integration points for each element. Thus, the S4 element demands more computing power and time.

Bridge piers and abutments were idealized using boundary conditions to represent the actual bearing used in the field. It was assumed that the piers and abutment would not be affected by the live-load (i.e., no settlement or side-sway) in the FE model. Pinned supports were modeled by restricting the translation in the x - and y -plane of the girder cross-section, whereas for roller supports, only the translation in the y -plane of the girder cross-section was restricted.

The FE model of the bridges consisted of multiple assemblies or parts (structure components). These parts needed to be joined together to construct the entire bridge structure. The joining of these parts was achieved by using constraint elements, or in this case a multi-point constraint (MPC). In ABAQUS, there are several predefined MPC, including BEAM and PIN. The BEAM MPC provides a rigid beam between two nodes to constrain the displacement and rotation at the first node to the displacement and rotation at the second node, corresponding to the presence of a rigid beam between the two nodes. It was mainly used for constraining the slab nodes to the stringer nodes for the composite action. For non-composite or zero-moment connections, such as the connection between the stringers and floor beams, PIN MPC was used. PIN MPC provides a pin connection between two nodes.

In addition to the node constraints, there were some members that share the same nodes (e.g., the diaphragms and the stringers). ABAQUS assumes a rigid connection if the same starting or ending node of two elements is used. Thus, to model the diaphragm connections, the rotation of the starting and ending nodes of the diaphragms (that were connected to the stringer) needed to be released. This was achieved by using the RELEASE commands specified by ABAQUS.

Mechanical Properties

The concrete properties were obtained using prediction equations provided by various American Concrete Institute (ACI) reports. The reasons for using predicted values rather than actual values were that there were no laboratory test data and no test standard or provision for testing concrete during its plastic stage. The concrete compressive strength at any age was predicted using the equation recommended by ACI Committee 209 (2008) report, which is given by

$$f'_c(t) = \frac{t}{a + t} f_{c,28} \quad (1)$$

where a and β are constants that can be determined through laboratory experiment, $(f_c)_{28}$ is the 28-day compressive strength of concrete and t is the age of concrete in days. Several HPC mixtures (Nassif et al., 2007) containing various combinations of pozzolans and slag cement were used for the determination of a and β . In this study, the values for a and β that were used were 4 and 0.85, respectively.

The modulus of elasticity was then determined by using ACI Committee 318 (2011) Building Code Requirement for Structural Concrete equation by

$$E = 0.033 w_c^{1.5} \sqrt{f'_c}, \text{ MPa} \quad (2a)$$

$$E = 33 w_c^{1.5} \sqrt{f'_c}, \text{ psi} \quad (2b)$$

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where f'_c is the compressive strength. The tensile strength was assumed to be 10% of the compressive strength, which matched quite well with the experimental data. Table 1 summarizes the mechanical properties used in the FE model.

Load Simulation

The load simulation was conducted to simulate concrete deck rehabilitation or replacement processes to determine whether traffic vibration, loads, and/or frequency have any effects on the concrete deck as it cures, specifically at 8 hours when the concrete remains plastic and is not fully hardened. Vehicular live loads were modeled using concentrated loads, which represented real truck traffic on typical highway bridges, that were obtained from various weight-in-motion (WIM) sites in New Jersey. These load data were collected by Nassif et al. (2010a). In the case where the truck could not be directly placed over the nodes, equivalent nodal loads were computed and used in the FE program. From the WIM data (Nassif et al, 2010), classes 6 (i.e., three axle, single unit) and 8 (i.e., four or less axle, single trailer) vehicles were determined to be the most frequent heavy trucks, and therefore, they were used in the simulation. The class 6 and 8 vehicles consisted of a 222 kN (50.0 kips) three-axle dump truck and a 350 kN (78.7 kips) four-axle single trailer truck.

Various computer runs were performed to simulate scenarios of truck traffic in various lanes and traffic directions. Other cases with trucks traveling side-by-side in other lanes were also considered. In order to derive data describing the behavior of a typical staged deck-replacement operation, models were developed to monitor the stresses in the deck under different live load configurations which would be normally encountered in urban work zones. These loading sequences are shown in Fig. 2. The loading sequences were used to simulate: (a) casting of the middle portion of the deck; (b) casting of the end portion of the deck; and (c) casting of the end portion of the deck but with one immediately adjacent lane closed to truck traffic.

RESULTS

High Performance Concrete Properties

Table 1 summarizes the concrete properties used in the FE model. These properties were based on mix designation G2M2 mixtures reported by Nassif et al. (2007). This mixture was chosen for the analysis because it represented a typical concrete-deck mixture with a specified compressive strength of 31 MPa (4,500 psi). The G2M2 mixture has a water-to-cementitious material ratio of 0.40 and it contained a combination of silica fume and slag cement with a 4% and 30% portland cement replacement by weight, respectively. The mixture also contained set-retarding admixtures, which affect its strength gain. Based on equation (1), the 8-hour compressive strength and cracking strain are 2.72 MPa (394 psi) and 35 microstrain, respectively.

Model Validation

To ensure that the modeling techniques used were accurate, the results were compared to field data obtained from Nassif et al. (2010b) who instrumented a bridge deck with vibrating-wire strain gages to monitor the deck strain over time. Figure 3 illustrates the comparison of field-measured concrete shrinkage with the FE model. The FE modeling techniques were also compared with field-testing data reported by Nassif et al. (2010a). Figure 4 illustrates, for girder strain under live load, the comparison between the measured data and the FE model. As can be seen in both figures, the modeling techniques, adopted to perform the analysis on the impact of adjacent traffic on the serviceability of bridge deck, provided good correlation with the field measured data. Thus, the FE model was used to performed parametric study to determine the effect of adjacent lanes traffics on bridge decks cracking.

Loading Sequences

As mentioned earlier, three loading sequences (Fig. 2) were evaluated. Figures 5a, 5b, and 5c illustrate the contour plots of the deck strain for load cases 1, 2, and 3, respectively. In the figures, red ovals highlight the locations of maximum average strain. For load case 1, the maximum tensile strain observed was 25 microstrain, which is approximately 29% lower than the cracking capacity of fresh concrete tested in the laboratory (35 microstrain at 8 hours). However, in reality higher maximum tensile strain could be recorded since there are overloaded trucks (trucks carrying more than the legal load). In fact, based on a report by Nassif et al. (2010a), the mean rate of overloading is 8.3%. Moreover, the dynamic impact factor would also increase the maximum tensile strain especially during construction when more road roughness is expected. For load case 2, a significantly higher maximum tensile strain is observed with a value of 47 microstrain, which is higher than the 8-hour cracking strain. Furthermore, there are large bands of concrete experiencing tensile strain higher than 15 microstrain, which could eventually lead to cracking. Load case 3 is similar to load case 2, but with one lane immediately adjacent to the new concrete closed to truck traffic. In this case, the maximum tensile strain observed was 23 microstrain. However, unlike cases 1 and 2, there are not too many bands of fresh concrete slab experiencing a strain greater than 10 microstrain. In fact, the critical strain observed was only between 5 to 8 microstrain. This leads to the conclusion that closing the lane immediately adjacent to the new concrete slab is warranted to ensure a crack-free slab.

Bridge Types

As mentioned earlier, two bridge types (slab-on-girder and through-girder bridges) were evaluated. The slab-on-girder represents typical short to medium span highway bridges and through-girder represented longer span bridges. Figures 6 and 7 illustrate the new-concrete deck strain along the length of the bridge for the slab-on-girder and through-girder bridges, respectively. The bridges were loaded using the worst-case scenario, load case 2, where the lane immediately adjacent to the new concrete slab was loaded. As anticipated, the long span through-girder bridge experiences higher tensile strain, leading the concrete to crack at multiple locations, but the slab-on-girder bridge only experiences high tensile strain near the pier or the position of maximum negative moment. For the through-girder bridge, there are multiple floor beams supporting the stringers and these regions are more prone to cracking. For the slab-on-girder bridge, the high tensile-strain location is due in part to the negative moment at the support.

Concrete Properties

Unlike OPC, the use of chemical admixtures such as a retarder is necessary for HPC mixes to delay the setting time of the concrete. As a result, the early-age properties, particularly the tensile strength and modulus of elasticity, are diminished during the first 24 hours. Figures 8a and 8b illustrate the effect of early-age properties on bridge-decks cracking using the FE model. Two 8-hours compressive strengths (904 psi and 1130 psi) were used in the analyses. As can be seen in the figures, there is a distinction between the two 8-hours compressive strengths. They indicate that there is higher potential for bridge deck cracking if the 8-hours compressive strength falls below 1000 psi.

CONCLUSIONS

Based on the analyses performed using the FE-model and the results of this study, the following conclusions and recommendations could be made:

- 1) There is high potential for cracking in the fresh HPC deck due to the chemical admixture that has a retarding effect. The model simulation shows that the HPC deck will undergo tensile stresses that can be larger than the early-age strength of the fresh concrete especially between 5-8 hours after casting. The structural behavior of the through-girder bridge is such that the stringers act as continuous beams over elastic supports (i.e., floor beams) and are susceptible to negative moment. Moreover, the differential deflection induced transversely by truckloads traveling in adjacent lanes will also cause camber in the stringers. The fact that concrete deck above the

Impact of Adjacent Lane Traffic on the Serviceability of Concrete Bridge Decks

- stringers will have tension stresses suggests that, at early age, cracks can be initiated first at these locations. This conclusion has been confirmed by the simulation of two side-by-side dump trucks.
- 2) Removing all chemicals used in the HPC mix that have any retarding effects on the concrete chemical reaction and bond development, especially at early ages, can alleviate bridge deck cracking.
 - 3) Temporarily closing the lane immediately adjacent to the new HPC deck slab can also alleviate or minimize bridge deck cracking. This can be achieved by ensuring that all trucks travel in the left lane using traffic signs at various stages of the roadway prior to arrival at the bridge site. This alternative does not infringe on the traffic flow or density, but rather ensures that multiple-presence events (e.g., having two heavy trucks side by side) would be rare or non-existent during the days following a concrete pour. The length of time, during which this option will be needed after a concrete pour to ensure a crack-free concrete deck, is dependent on the strength and stiffness development of the concrete.
 - 4) It is recommended that the HPC should have at least a minimum of 2000 psi compressive strength at 1 day or more specifically 1000 psi at 8 hours to minimize the potential for cracking of bridge-decks due to transverse differential deflection induced by truck traffic in adjacent lanes.

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Table 1—Mechanical Properties of the Concrete Deck

Time Days	Compressive Strength MPa (psi)	Tensile Strength MPa (psi)	Modulus of Elasticity GPa (ksi)	Cracking Strain $\mu\epsilon$
0.33	2.72 (394)	0.27 (39)	7.80 (1131)	35
0.5	3.98 (577)	0.40 (58)	9.44 (1370)	42
1	7.50 (1087)	0.75 (109)	12.96 (1879)	56
3	21.66 (3142)	2.17 (314)	22.03 (3195)	73
7	30.44 (4415)	3.04 (442)	26.11 (3787)	95
14	35.47 (5145)	3.55 (515)	28.19 (4089)	110
28	35.24 (5111)	3.52 (511)	28.10 (4075)	126

Impact of Adjacent Lane Traffic on the Serviceability of Concrete Bridge Decks

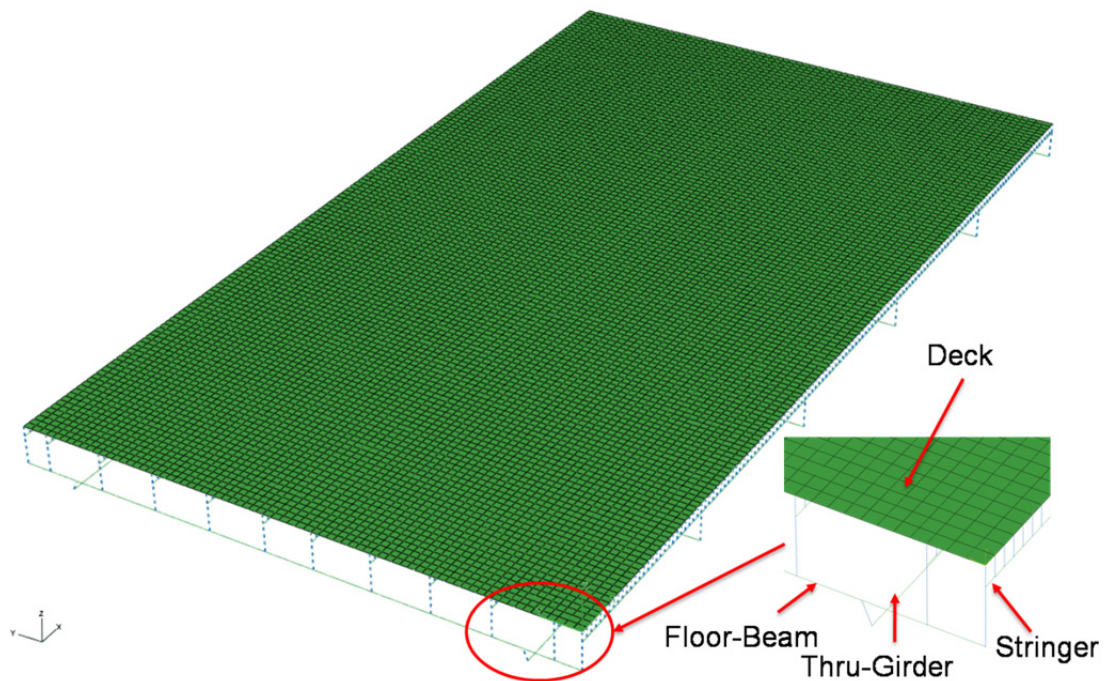


Fig. 1—FE Model of A Typical Girder Bridge with Floor Beam

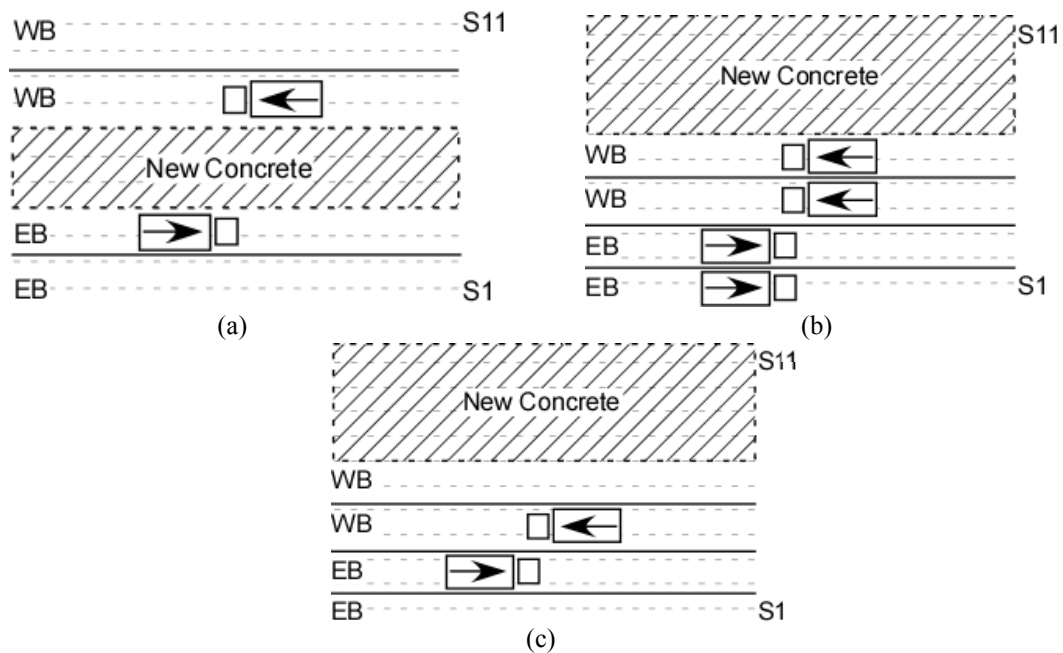


Fig. 2—Loading Sequences: (a) Case 1; (b) Case 2; and (c) Case 3

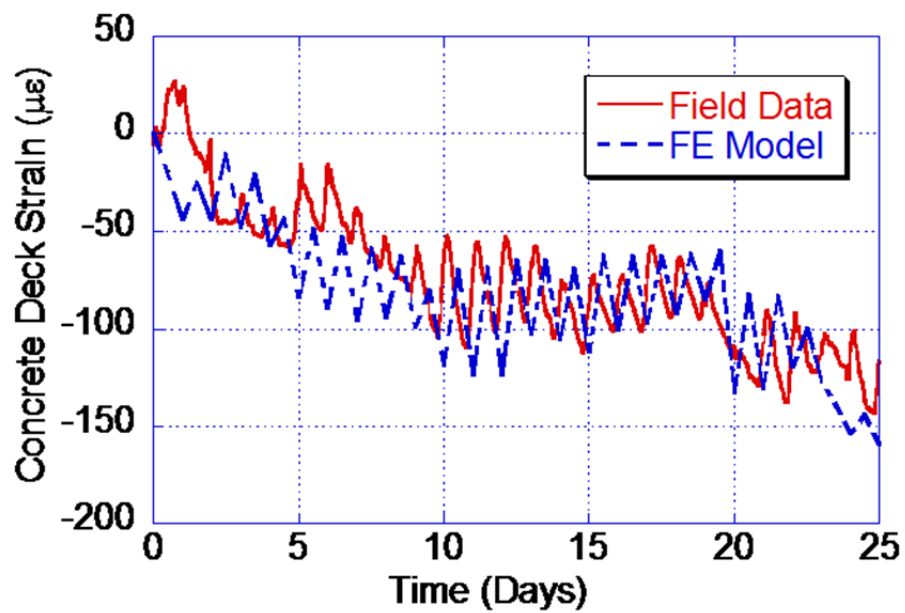


Fig. 3—Comparison of Field Measured Concrete Deck Shrinkage with FE Model.

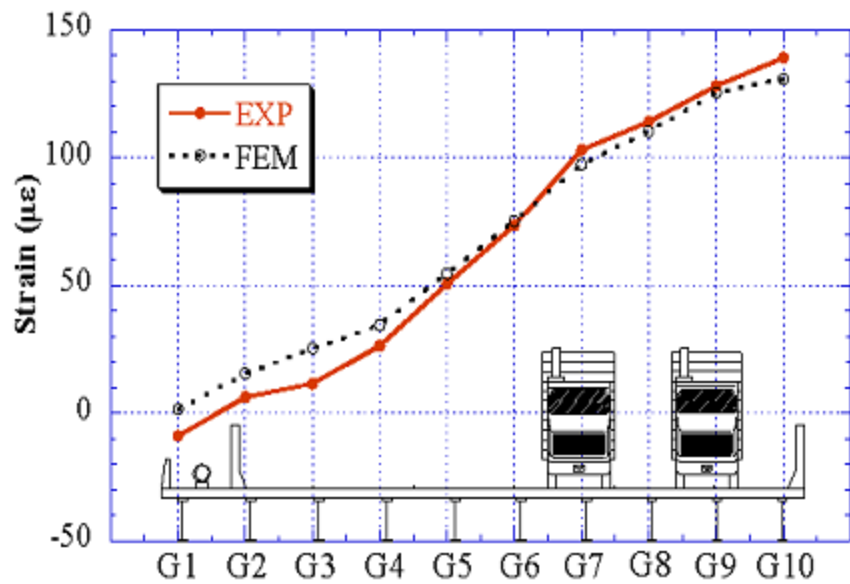


Fig. 4—Comparison of Field Measured (EXP) Girder Strain with FE Model (FEM).

Impact of Adjacent Lane Traffic on the Serviceability of Concrete Bridge Decks

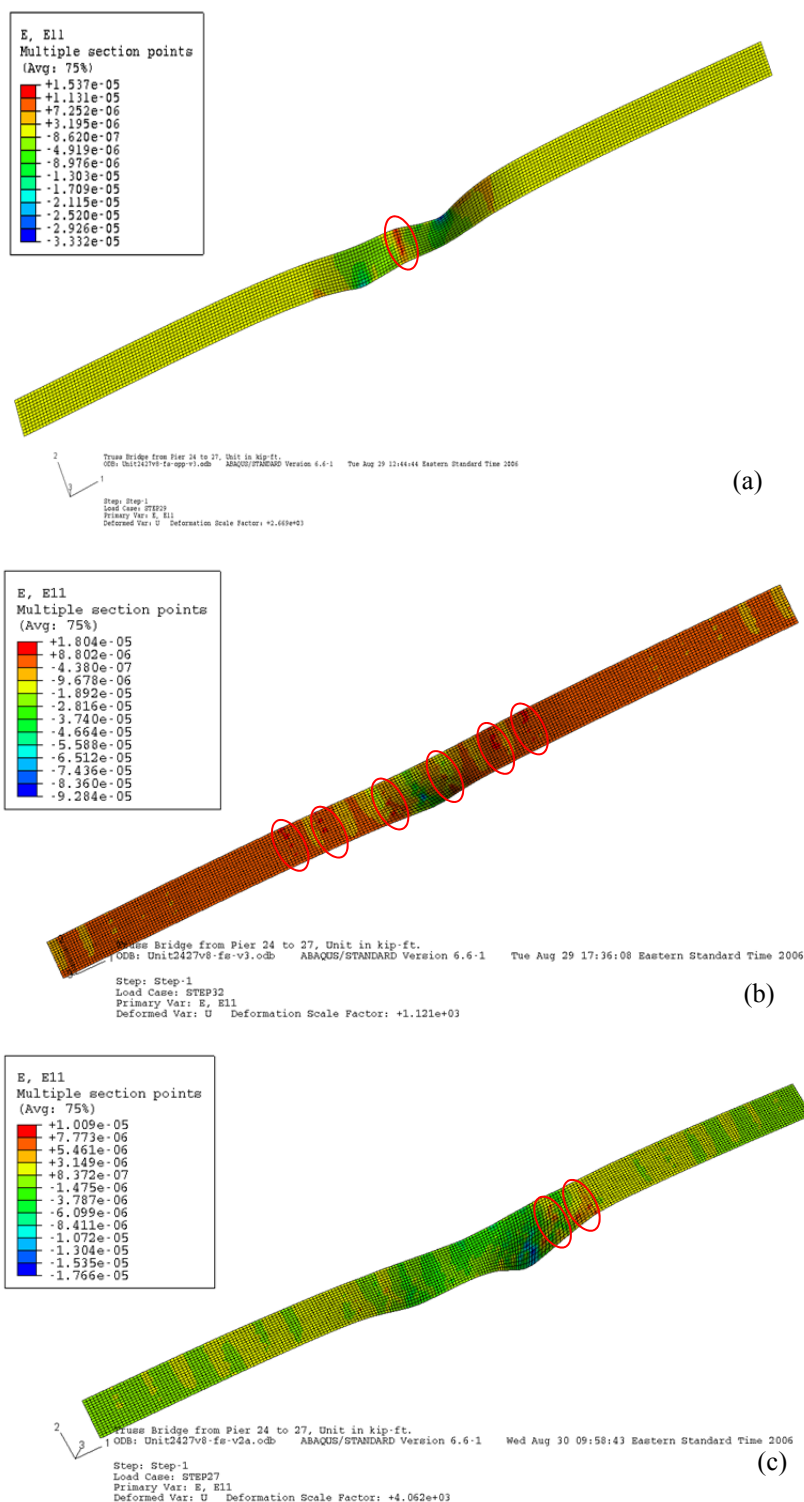


Fig. 5—Contour Plot of Concrete Strain for Loading Sequences: (a) Case 1; (b) Case 2; and (c) Case 3.

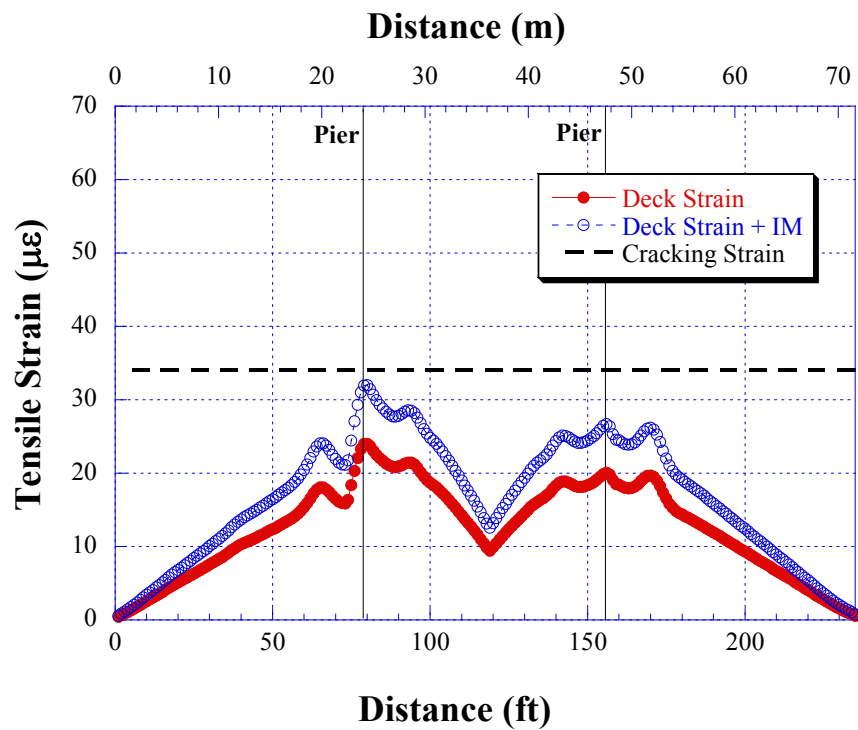


Fig. 6—Concrete Deck Strain of a Slab-On-Girder Bridge

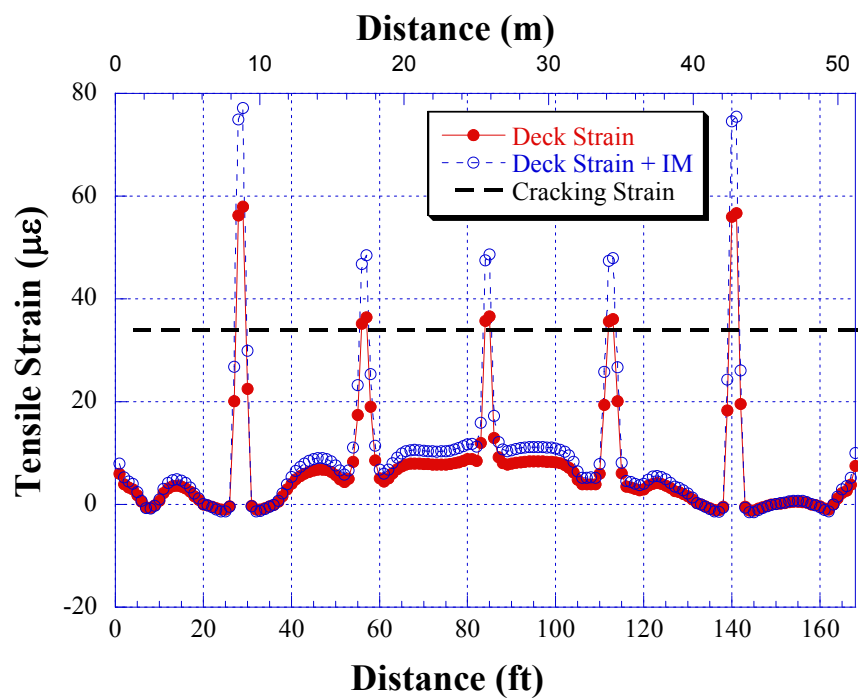
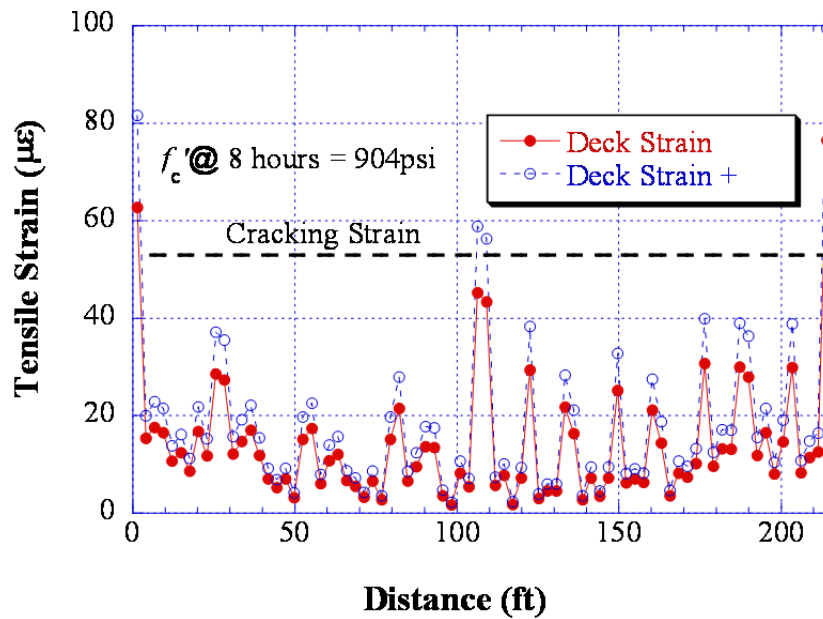
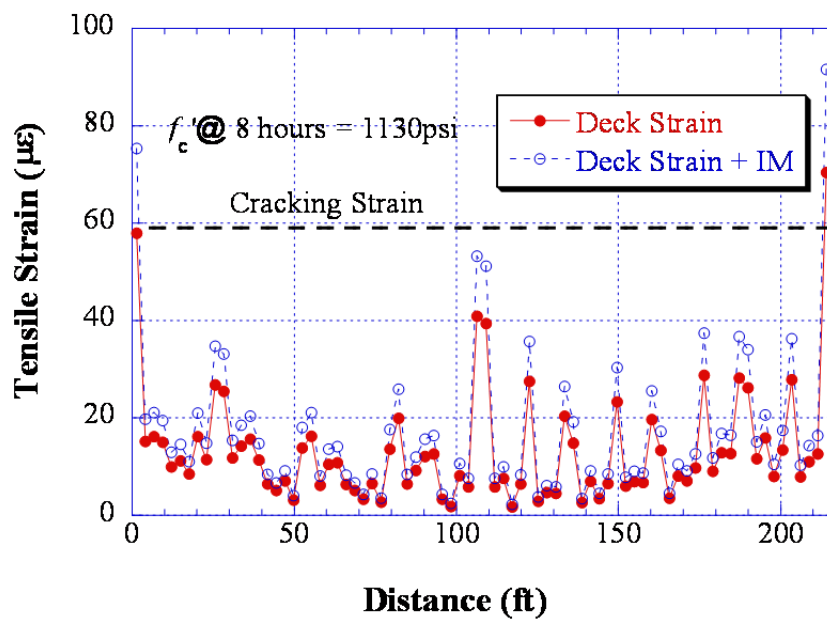


Fig. 7—Concrete Deck Strain of a Through Girder Bridge

Impact of Adjacent Lane Traffic on the Serviceability of Concrete Bridge Decks



(a) 8-Hours Compressive Strength of 904 psi.



(b) 8-Hours Compressive Strength of 1130 psi.

Fig. 8—Effect of Early-Age Properties on Bridge-Deck Cracking

Design Improvements from Structural Investigations

Jared E. Brewe and W. Gene Corley

Synopsis: The Oklahoma City Bombing is used as a case study to demonstrate how structural damage investigations can lead to code changes and improvements in design. This paper describes the use of seismic detailing to improve blast resistance. Such detailing has been incorporated into the soon to be published ASCE/SEI Standard on Blast Protection of Buildings.

Keywords: blast loading, progressive collapse, failure investigation

Brewe and Corley

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INTRODUCTION

On April 19, 1995, the Murrah Federal Office Building in Oklahoma City was attacked by a terrorist bomb blast. The nine-story portion of the building sustained significant damage from the blast which resulted in progressive collapse of additional, previously unaffected sections of the building. The bombing raised awareness on the effects of blast hazards and terrorist attacks on buildings, the threat of progressive collapse of building systems, and highlighted the need to improve standards to prevent catastrophic failure.

The Federal Emergency Management Agency (FEMA) deployed a building performance assessment team to investigate the damage. *FEMA 277: The Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation* [1], documented the investigation, reported on the causes of the collapse, and provided strategies for mitigation to prevent the recurrence of this type of damage. The background information contained in this paper along with some figures have been gleaned from the FEMA 277 report [1]. The focus of this study is on the performance of the Murrah Building, the use of seismic detailing to provide improved resistance to bombing, and discussion of subsequent changes to codes and standards.

ALFRED P. MURRAH FEDERAL BUILDING

The Murrah building project consisted of a nine-story office building (hereafter referred to as the Murrah building) along with one-story ancillary east and west wings and an adjacent multi-level parking structure south of the office building, partially below grade and partially above grade. This project was designed in the early 1970's and construction was completed in early 1976. Following the attack, the investigation focused on the nine-story portion of the Murrah building. Primary effects of the blast occurred along the north face, where the bomb was located at street level. The nine-story and one-story portions were demolished shortly after the blast and completion of investigations. The parking structure was barely damaged and was not demolished. Figure 1 shows the Murrah building prior to the attack.

Details of Murrah Building

The nine-story structure of the Murrah Building was an Ordinary Moment Frame of reinforced concrete supported on columns. Overall plan dimensions were approximately 220 ft (67.1 m) in the east-west direction and approximately 100 ft (30.5 m) in the north-south direction, shown in Figures 2 and 3. In plan, the structure for the nine-story portion consisted of ten 20 ft (6.1 m) bays in the east-west direction and two 35 ft (10.7 m) bays in the north-south direction. Shear walls and other localized columns and walls were in the core area at the midpoint of the south side of the building. Four large, prominent, vertical circular tube columns, one at each of the four corners of the building, acted as air intakes/exhaust shafts for the ventilation system. Typical floor-to-floor height was 13 ft (4.0 m) for floors 1 through 8 with 14 ft (4.3 m) for the ninth floor.

Design Improvements from Structural Investigations



Figure 1: Murrah Building prior to Attack

Structural drawings showed that the Murrah Building consisted of cast-in-place ordinary reinforced concrete framing, designed in accordance with ACI 318-71 [2]. The building system included conventionally reinforced columns, girders, beams, slab bands, and a one-way slab system, with exterior spandrels supporting the exterior curtain-wall. The lateral load resisting system for wind forces was composed of reinforced concrete shear walls located within the stair and elevator area in the south-central portion of the building. A significant feature of the structural system was a transfer girder spanning east-west on the north face of the building at the third-floor level. This girder is visible just below the curtain wall in Figure 1 and along column line G in Figure 3. The transfer girder was used to support intermediate columns from above resulting in a column spacing of 40 ft (12.2 m) for the first two floors along the exterior column line. In addition, the curtain wall system at this location was set back several feet, providing an open space on the first two levels as shown in Figure 2.

Plans showed that the design live loads followed requirements of the Oklahoma Building Code. Wind loads were included but no blast or earthquake loads were required. Although neither the governing building code nor the owner required consideration of blast loading or earthquake loading, the required wind-load resistance provided substantial resistance to lateral load.

Columns located along the north face and supporting the transfer girder were 20 in (508 mm) wide by 36 in (915 mm) deep. They were reinforced with 20 #11 (No. 36M) vertical bars and two #4 (No. 13M) horizontal ties spaced at 16 in (406 mm) on center. These columns extended from ground level to the third floor transfer girder and were connected to the building at the second floor by a transfer strut. However, the columns were laterally unbraced in weak axis bending for a 21 ft (6.4 m) height between ground level and the third floor.

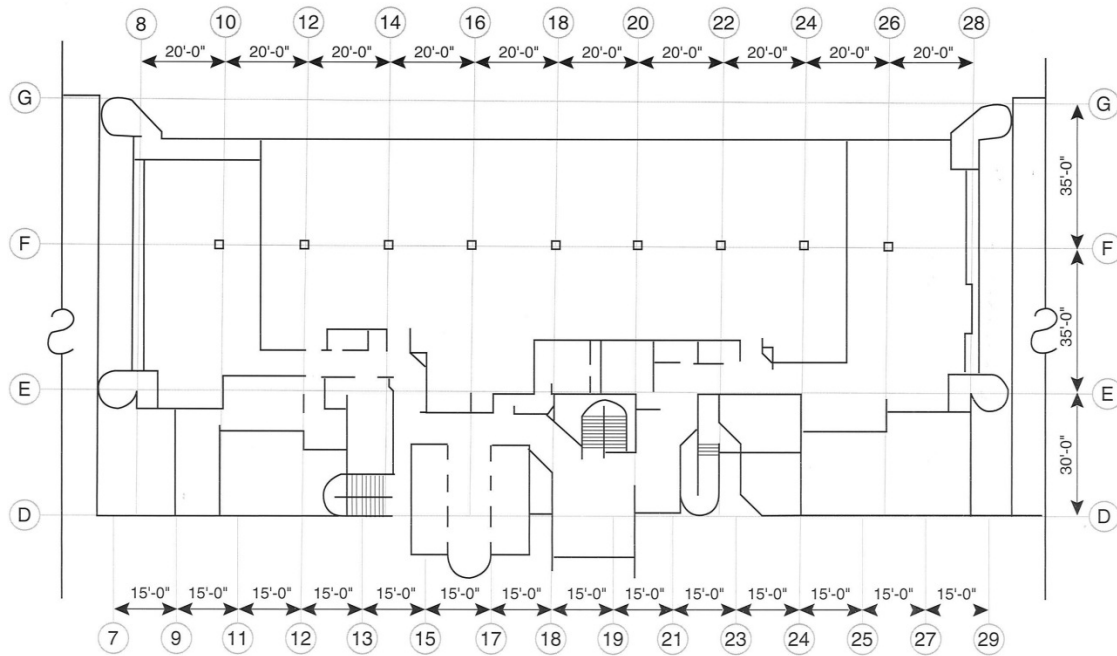


Figure 2: Floor Plan of Second Floor of Murrah Building

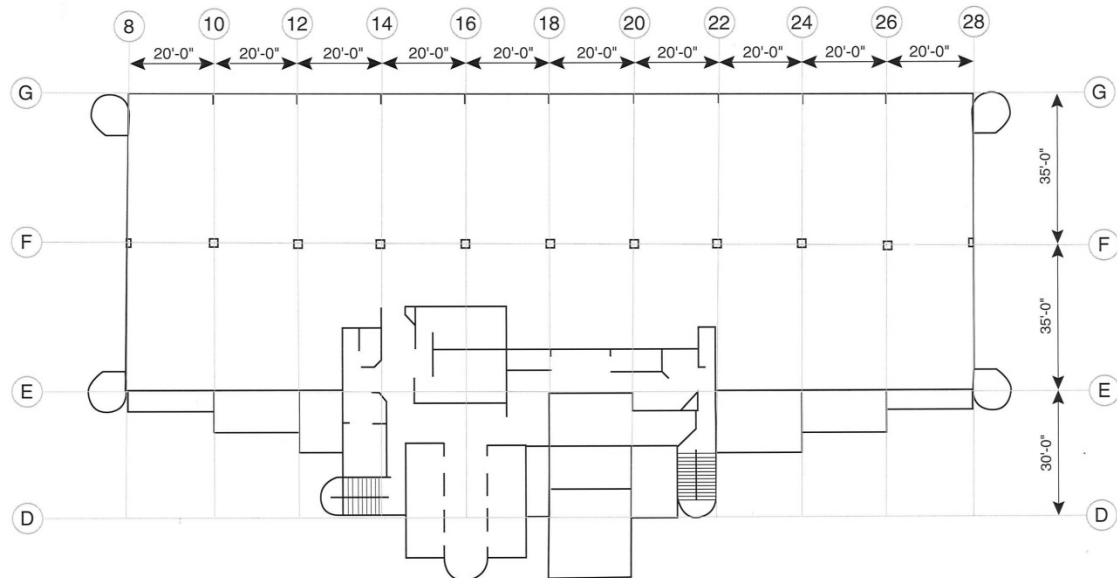


Figure 3: Floor Plan of Third Floor of Murrah Building

The transfer girder was 36 in (915 mm) wide and 60 in (1500 mm) deep, spanning the entire north face. Top reinforcing consisted of three continuous #11 (No. 36M) top bars, spliced near mid-span, the length of the building along with eight #11 (No. 36M) bars placed over the columns in the negative moment regions. Bottom reinforcing for the three interior spans consisted of ten #11 (No. 36M) bars, with four of the bars extending the full span to the column centerlines. Exterior spans contained nine

Design Improvements from Structural Investigations

#11 (No. 36M) bars with three extending to the column centerlines. In each span the bottom reinforcing was terminated at the column centerline. Consequently, there was no continuous reinforcement in the transfer girder or spandrels of the building as would be required by current structural integrity design standards. It is noted that design standards at the time the building was built did not require integrity reinforcement and that analyses show that this building met or exceeded the design code requirements in every way that was checked.

One-way slabs spanned between beams that framed perpendicular to the transfer girder. The slabs were 6in. (152 mm) deep and reinforced with #4 (No. 13M) bars spaced 9in. (228 mm) on center in the east-west direction as primary reinforcement. In addition #4 (No. 13M) bars spaced 18in. (457 mm) on center were provided in the north-south direction as temperature reinforcement. Both layers of reinforcing were placed in the bottom of the slab to resist the tension stress under gravity loading.

According to general notes on the structural drawings, the Murrah Building project was all reinforced concrete that was fabricated in accordance with ACI 318-71 code requirements. Yield stress for ties, #3 (No. 10M) bars, and stirrups was 40,000 psi (280 MPa). Yield stress for all other deformed bars and all welded wire fabric was 60,000 psi (420 MPa). The 28-day concrete compressive strengths included 3,000 psi (21 MPa) for foundation and equipment bases and 4,000 psi (28 MPa) for structural beams, slabs, columns, walls, counterforts, pilasters, spread footings, and parking garage exterior walls. General notes also required all reinforcing bar splices to be lapped 30 bar diameters unless otherwise noted. Testing of concrete cores and reinforcing bars taken from the debris indicated that the strengths were equal to or greater than the minimum specified and easily met the design specifications.

Blast Size and Loading

Forensic engineering techniques were used to estimate the blast size and to provide a means for assessing blast response of the building and potential mitigation techniques. Based on observations made of the crater and other damage, the blast that damaged the building had a yield equivalent to approximately 4,000 lbs (17.8 kN) of TNT [1].

Estimated location of the explosive-laden truck and the resulting crater along the building north elevation are shown in Figures 4 and 5. The explosion was centered approximately 15.6 ft (4.8 m) from Column G20 (see Figure 5) and caused a crater approximately 28 ft (8.5 m) in diameter. The north face of the building next to the blast sustained the brunt of the explosive loading effects, with limited damage to the remaining structural components.

As the blast wave propagates radially from the point of origin, the relative amount of overpressure applied to the building components is reduced. Peak overpressure ranged from 10,000 psi (70 MPa) near the origin down to 9psi (62.1 kPa) at the furthest point on the north face. These overpressures correspond to an equivalent uniform overpressure of 140 psi (965 kPa). Higher peak overpressures correspond to a shorter duration of loading, while lower overpressures correspond to longer durations. The equivalent uniform duration for the uniform overpressure was assumed to be 5 milliseconds.

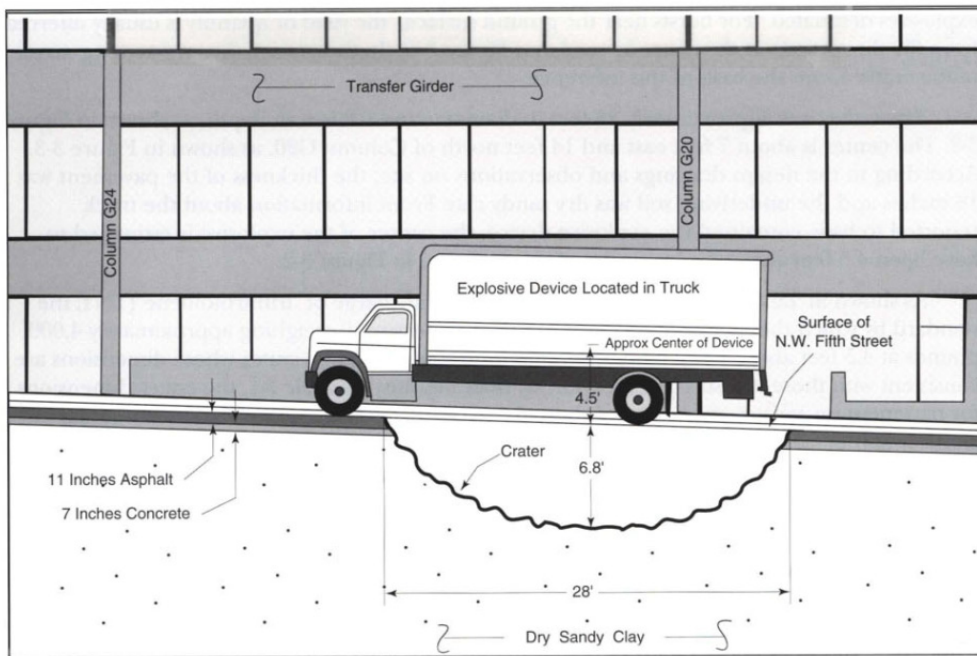


Figure 4: Location of Truck and Dimension of Crater

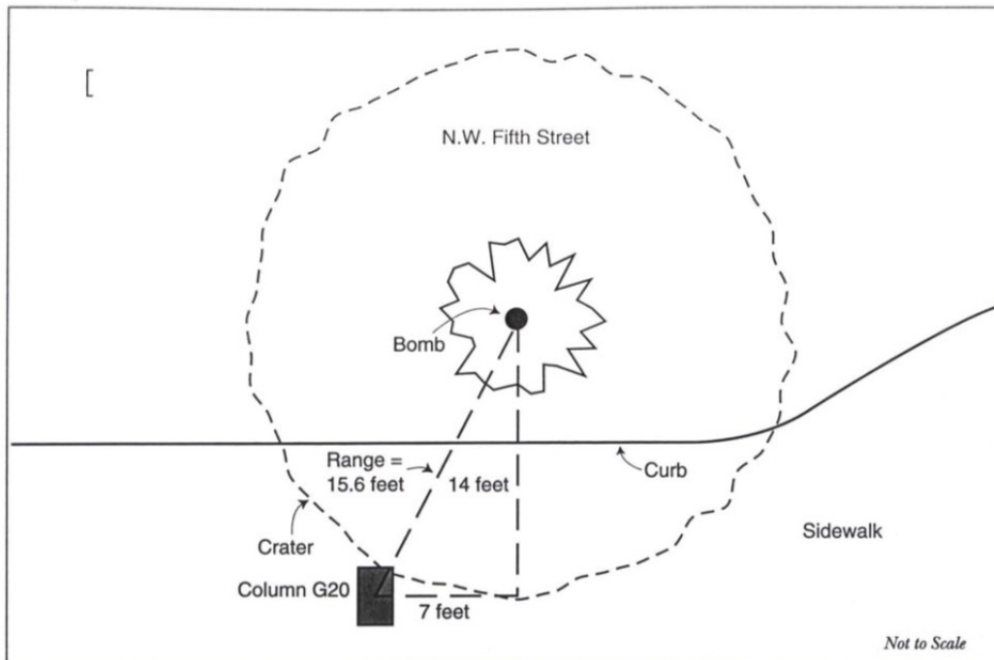


Figure 5: Center of Blast and Location of Crater

The blast loading applied to individual building components is assumed to be a short duration, uniformly distributed impulse pressure. Columns located on the north face are subjected to a lateral

Design Improvements from Structural Investigations

pressure over their height, typically in weak axis bending due to their location relative to the blast. The one-way floor slabs are subjected to an uplift pressure over their area.

The combination of high equivalent explosive weight and small standoff distance create incident overpressures much larger than typical blast resistant design would dictate. While actual blast resistant design overpressures used for public buildings are not publicly available, assumed pressures are typically in the range of 10 to 100 psi (69 to 689 kPa) depending on standoff distances.

Building Response to Blast

The extent of damage directly attributable to blast effects was limited to columns and one-way floor slabs in close proximity to the point of origin. Analysis of these members indicated that their failure was initiated due to the stress from the overpressures described above. Figures 4 to 7 identify the locations of the columns relative to the blast along the north face, which is column line G.

Examination of damage and subsequent blast loading calculations determined that a large portion of the column nearest the blast (G20) was destroyed by brisance (or shattering). Further analysis determined that the next column in each direction supporting the transfer girder (G24 and G16) most likely underwent a brittle failure in shear, just before reaching their flexural capacities.

Analysis of the blast effects on the one-way floor slabs in proximity to the blast identified that a number of them would most likely have failed in uplift, due to the lack of negative moment steel, as shown in Figures 6 and 7. The failure of each of these slabs should not lead to further damage since they do not support other structural members. However, their failure does reduce the stiffness of the overall structural system.

Progressive Collapse

The forensic investigation identified that progressive collapse (also known as disproportionate collapse) was the primary cause of the extensive damage observed in the Murrah building. Collapse occurred due to the failure of columns supporting the transfer girder. The loss of the first column (G20) resulted in the transfer girder being left without support for a length of 80 ft (24.4 m), with loss of the other two neighboring columns increasing the unsupported length to 160 ft (48.8 m). As discussed above, the transfer girder contained three #11 (No. 36M) continuous top bars but with no continuous bottom steel. Consequently, in areas above the columns, the girder was unable to develop tension forces in the bottom steel due to bar termination at the column centerlines. After losing support from three columns, the transfer girder was no longer able to support the columns and floors above. Consequently, approximately 50 percent of the total floor area of the building suddenly collapsed, resulting in a large number of casualties. The Murrah building following the blast and resulting collapse, is shown in Figure 8 and the approximate extent of collapsed floor area is shown in Figure 9.

The forensic structural analysis investigated the probability of progressive collapse of the moment frame due to column loss. Three different collapse mechanisms were assumed to develop, one without and two with removal of the column nearest the blast, as shown in Figure 10. Calculations indicated that Mechanism 2 is the controlling failure mode and results in an estimated capacity of approximately 60 to 70 psf (413 to 432 kPa) for self-weight plus live load. This capacity is less than the approximately 110 psf (758 kPa) self-weight plus live load present in the building at the time. Consequently, removal of any one column supporting the as-built transfer girder would cause failure of all of the floors above over a length of about 80 ft (24.4 m).

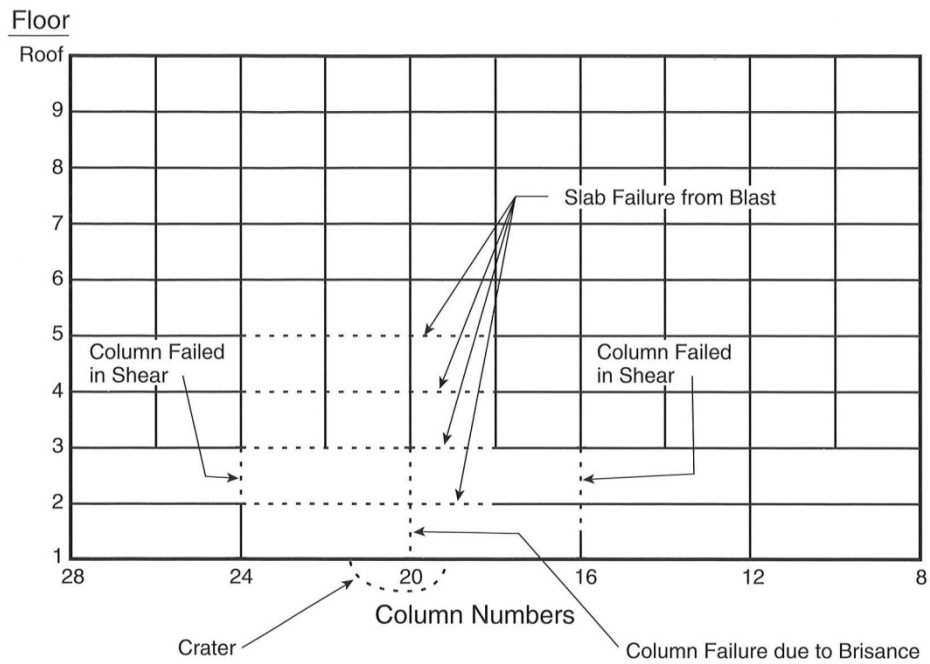


Figure 6: Schematic of Building Response to Blast, North Elevation

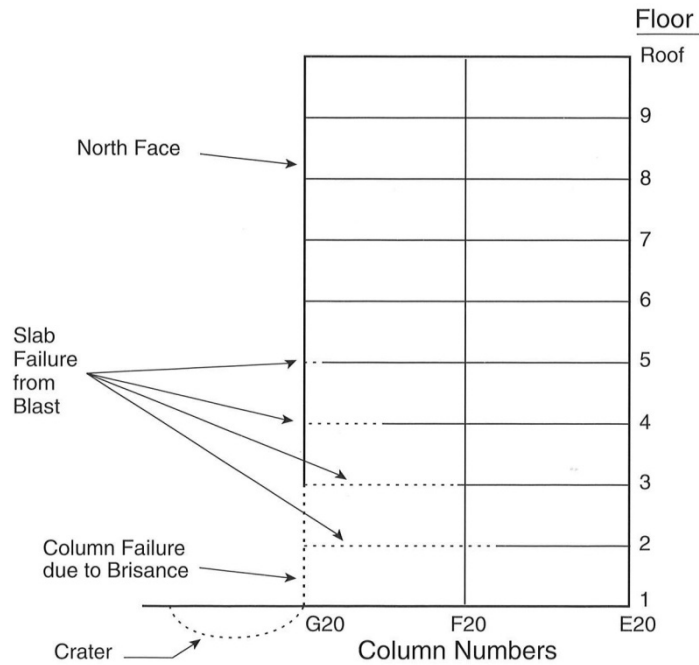


Figure 7: Schematic of Building Response to Blast, Cross-Section

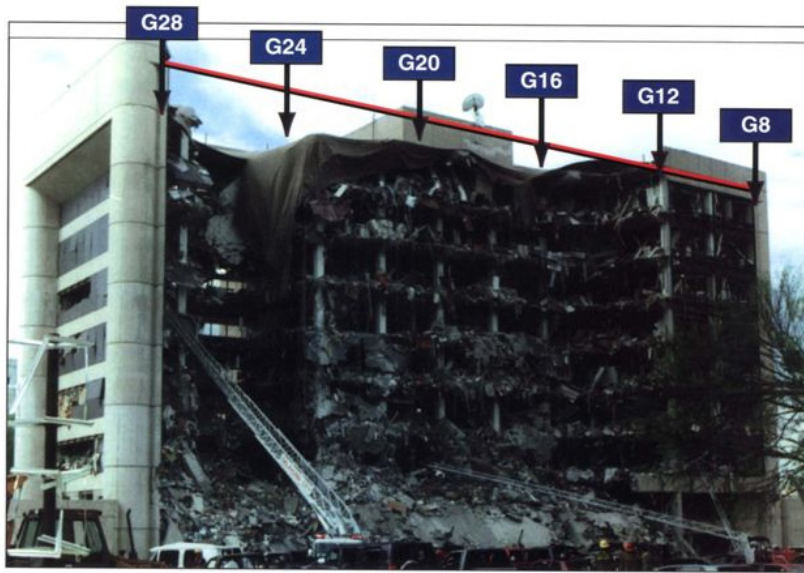


Figure 8: Murrah Building Following Blast and Collapse

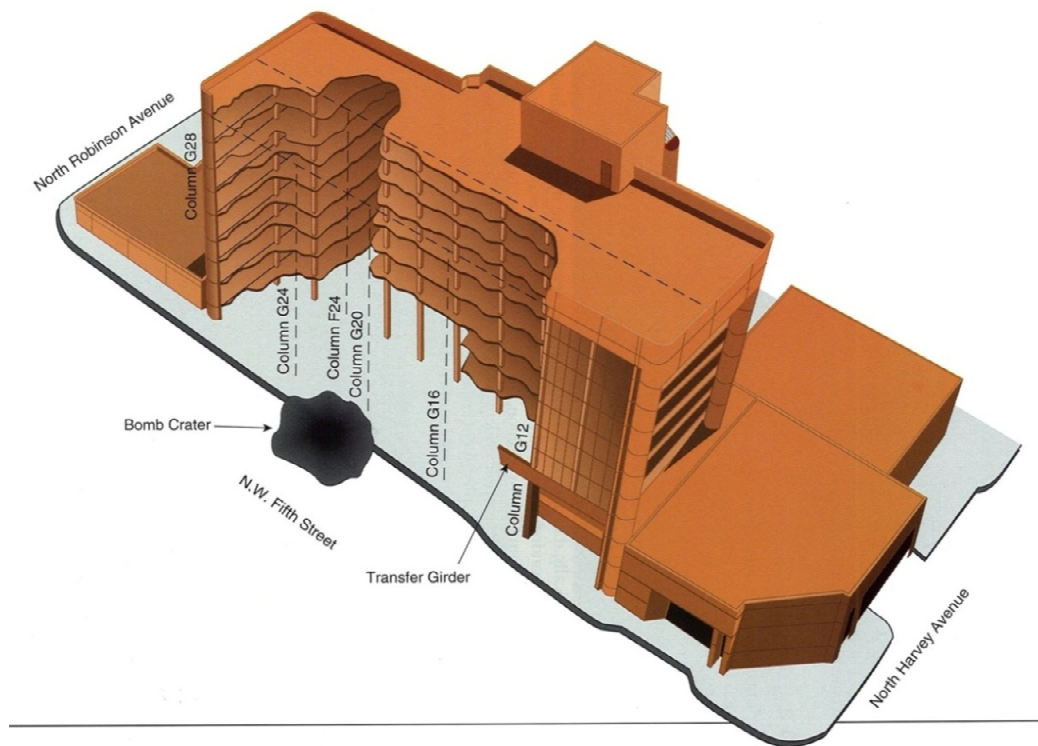
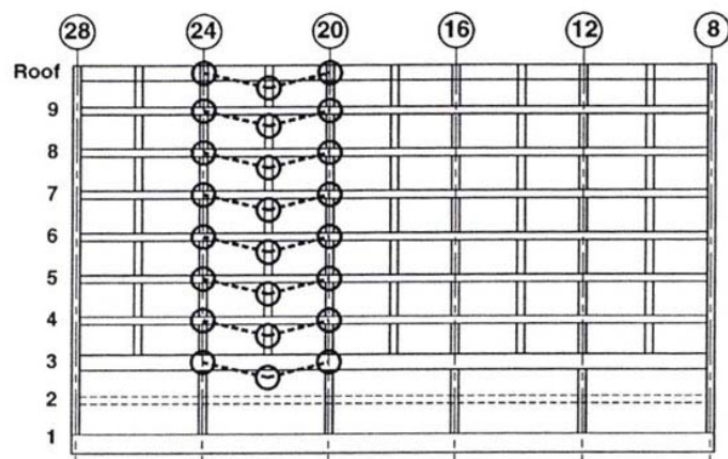
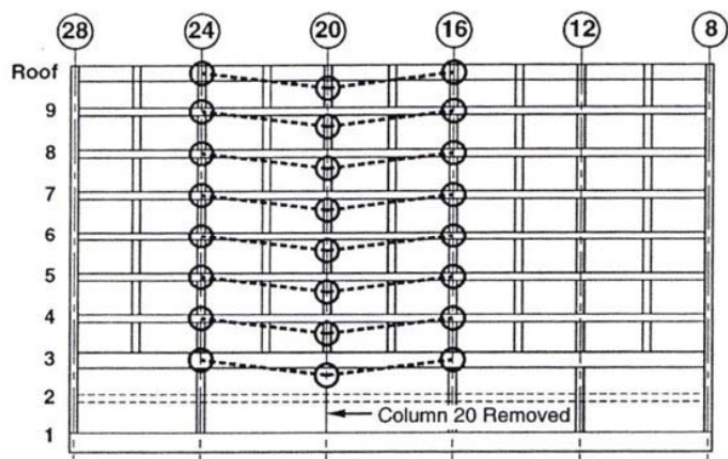


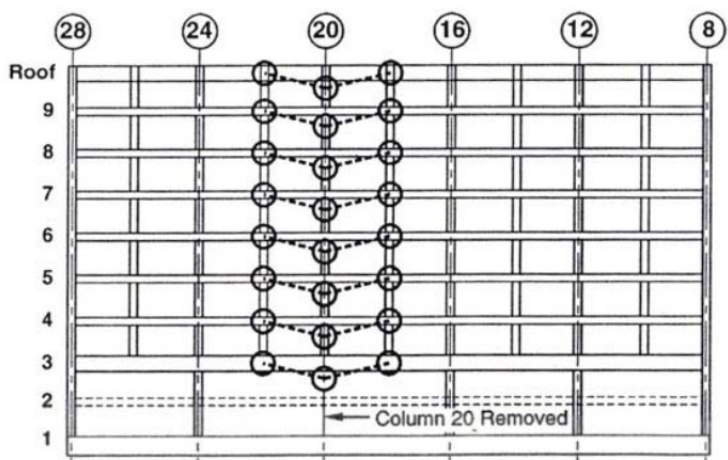
Figure 9: Extent of Floor Collapse



Mechanism 1



Mechanism 2



Mechanism 3

Figure 10: Assumed Possible Mechanisms

STRATEGIES FOR DAMAGE MITIGATION

At the time the Murrah building was designed and constructed, seismic design and detailing, integrity reinforcement, and full-capacity mechanical butt-splices were not recognized or recommended in governing design codes. The following discussion explores the performance of the Murrah building had these items been available and utilized at the time.

Seismic Detailing

Ordinary Moment Frames are not designed or detailed to provide ductility and energy dissipation during extreme loads such as earthquake and blast. At the time the building was designed, recommendations for the design and detailing for energy dissipation were not widely available. In addition, since the building was not designed for seismic forces, the design did not require special seismic detailing.

Intermediate and Special Moment Frames are structural systems with much higher capacity to dissipate energy. Details typically found in Special Moment Frames include continuity of top and bottom reinforcement, design shear strengths which are able to develop the maximum probable moments, and increased concrete confinement at locations of potential hinge mechanisms.

Shear Reinforcement – According to governing codes at the time the building was designed, seismic loads were not required for the building. Additionally, the seismic design requirements in ACI 318-71 [2] were only included as an appendix and had not yet been fully incorporated into the code. However, for special ductile frame columns (Special Moment Frames), Section A.6.6 requires that adequate transverse reinforcement be provided to develop shear capacity at least equal to the shears developed at the formation of plastic hinges in the frame, with a maximum spacing of half of member depth. Analysis has found that the amount of shear reinforcement, two #4 (No. 13M) bars at 16 in (406 mm) on center, provided for the columns would not meet the requirement in Section A.6.6 for transverse reinforcement if it had been required.

Integrity reinforcement – Integrity reinforcement requirements for beams-and-slab column intersections are not intended to transfer the full design stresses. Rather they provide a means to develop catenary action when the support has been removed. The purpose as defined in the ACI 318 commentary is “to improve the redundancy and ductility in structures so that in the event of damage to a major supporting element...the structure will have a better chance to maintain overall stability.” Integrity reinforcement is another requirement in seismic detailing to provide ductility.

Starting with the 1989 ACI 318 code, requirements for structural integrity reinforcement were included in Section 7.13 [3]. For perimeter beams, Section 7.13.2.2 requires that at least one-sixth of the negative moment reinforcement and one-quarter of the positive moment reinforcement be continuous and tied together with closed stirrups. For beams not on the perimeter, Section 7.13.2.2 requires one-quarter of the positive moment bars to be continuous. For two-way slab systems, Section 13.4.8.5 requires that two bottom bars in each direction be carried through columns continuously or anchored within the columns. This continuous reinforcement is able to develop tensile forces if a support is removed and provides a means to develop the catenary action within the structure.

The transfer girder included three #11 (No. 36M) continuous top bars, out of 11 total bars at the supports, which would have met the first requirement in Section 7.13.2.2. However, without the continuous bottom bars the tension forces are not developed and flexural cracking propagates until the top bars are essentially ineffective.

The reinforcement required for structural integrity can be provided by bars spliced at midspan for top reinforcement and at or near the supports for bottom reinforcement. Section 7.13 specifies Class A

splices, which are defined in Section 12.15.1. This section requires a minimum lap of one full bar development length. The intent is to splice these bars at locations of least demand under normal design loads, which is why there are specific requirements for top and bottom bars.

Full-capacity mechanical butt-splices – In regions of Special Moment Frames where flexural yielding is anticipated, ACI 318 does not permit the use of lap splices due to unreliable inelastic performance. Section 21.3.2.3 of the 1989 ACI 318 code defines these regions as within the joints, twice the member depth from the face of the joint, or where analysis indicates.

First recognized in the 1999 ACI 318 code [4], full-capacity mechanical butt-splices provide an alternative to lap splices in members resisting earthquake-induced forces. Specifically, Section 21.2.6 provides for the use of Type 2 mechanical splices to "develop the specified tensile strength of the spliced bar" at any location, including hinging regions. In view of that, Type 2 full-capacity mechanical butt-splices could connect integrity reinforcement to provide the tension force capacity.

Building Performance with Seismic Detailing

Using seismic detailing, structural analyses were performed to explore the performance of the building under several scenarios [1]. These scenarios include combinations where columns were designed to meet Special Moment Frame detailing and transfer-girder reinforcing was designed to meet the structural integrity requirements, including utilizing full-capacity mechanical butt-splices.

Due to proximity to the blast origin, it is believed that Column G20 would have been destroyed by brisance even with Special Moment Frame detailing. However, with heavy confinement reinforcement the chance of survival would have increased. Assuming the survival of Column G20, structure loss would be limited to floor slabs destroyed directly by the air blast, with a reduction in loss of approximately 85 percent.

Columns G16 and G24 failed in shear prior to reaching their full moment capacity as discussed above. If Special Moment Frame details including transverse reinforcement had been incorporated, it is likely that these columns would have formed a hinge mechanism prior to shear failure. Subsequently, G16 and G24 would not have failed abruptly due to the blast loading if Special Moment Frame detailing had been used. Assuming that Column G20 was removed by the blast and failure of the transfer girder occurred in the spans between Columns G16 and G24, structure loss would be limited to those floors destroyed by air blast and those spans located between Column Lines 16 and 24. The resulting loss would be less than 50% of that without Special Moment Frame detailing. Additionally, if structural integrity reinforcement was provided in the transfer girder, the probability of collapse of the floors above would be reduced. Consequently, destruction would have been limited to those areas destroyed directly by the air blast, and estimated losses would have been as small as 20% of the losses that actually occurred. It is likely that the building would no longer be structurally safe, but collapse would have been prevented.

Assuming a best case scenario with all bottom transfer-girder reinforcing continuous, a significant increase in the capacity of Mechanism 2 (Figure 10) would be realized. Analysis indicated that even if column G20 had been completely removed, there would be a low probability of building collapse due to development of a combination of Vierendeel, catenary, and transfer girder flexure. Although collapse of the building frame would have been prevented, several of the floor slabs would still be destroyed as a direct result of the air blast. Consequently, the reduction in casualties and in catastrophic damage to the building would not have been 100 percent, but is estimated to be approximately 80 percent.

CODE CHANGES FROM MURRAH BUILDING

Following the Murrah building bombing and other terrorist attacks on public facilities, government departments have worked to develop and update design standards and guidelines to protect their

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occupants, employees, and the general public. Significant interest had been paid to prevention of progressive collapse due to the loss of a support or other primary frame member.

Prior to this attack, different government organizations developed and specified individual criteria for the design of buildings against blast. Specifically the Department of Defense (DoD), Department of State (DoS), Department of Energy (DoE) and General Services Administration (GSA) each had specific criteria which they required for their facilities. Following this and other attacks, there has been an attempt to consolidate and combine the criteria to develop more uniform recommendations for blast protection under a United Facilities Criteria (UFC). While this is an ongoing process, some of the design guides and standards which have been produced are:

- Interagency Security Committee, *Physical Security Criteria for Federal Facilities*, Department of Homeland Security. (For Official Use Only)
- UFC 3-340-01: Design and Analysis of Hardened Structures to Conventional Weapons Effects (For Official Use Only)
- UFC 3-340-02: Structures to Resist the Effects of Accidental Explosions
- UFC 4-010-01: DoD Minimum Antiterrorist Standards for Buildings
- UFC 4-023-03: Design of Buildings to Resist Progressive Collapse

Updates to these standards since the Murrah bombing have included the seismic detailing strategies noted above. These criteria are typically designed to provide resistance to a specific threat.

A new document developed from this tragedy was the *GSA Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* [5], first published in November 2000 and later updated in June 2003. This publication was developed to provide guidance for the reduction of progressive collapse, independent of threat. The GSA guidelines do not require and are not a substitute for blast resistant design; instead they provide a method to determine the progressive collapse resistance of new and existing facilities. In Chapter 4 on Reinforced Concrete, several provisions are similar to those of the findings from the Murrah building collapse. Guidance is given on the use of detailing to provide structural continuity and ductility, analogous to the structural integrity requirements discussed above. Designing for member capacity to resist shear prior to flexural failure is highlighted. The analysis for progressive collapse is focused on regular shaped buildings and requires analysis of removal of a column on both the long and short exterior dimensions, as well as at a corner. If underground parking is available, removal of an interior column is also required. For load bearing wall structures, a similar analysis is required for removal of a wall section along each face and at corners. Many of the requirements in these guidelines relate to the collapse of the Murrah building.

While terrorist threats are typically assumed to target high profile public facilities, private building owners and occupants are becoming increasingly aware of the risks. The soon to be published ASCE/SEI Standard on Blast Resistant Design [6] provides a document that private building owners can use for their buildings. The standard relies heavily on the requirements of UFC 3-340-02, but also improves on those requirements when needed.

ASCE/SEI Blast Standard

Chapter 6 of the ASCE/SEI Standard provides requirements for design and analysis of structural systems. Section 6.2.3.4.4 covers the requirements of design to resist progressive collapse, with the commentary to this section including a comprehensive discussion of progressive collapse in the context of blast resistant design. The primary goal is to improve ductility and redundancy of the structural system. Section 6.3.1.3 specifies that Concrete Moment Frame Systems have adequate confinement and that sufficient shear capacity is provided to develop plastic hinging and ductile inelastic deformations.

Chapter 9 contains the requirements for material specific detailing, with Section 9.2 specific to concrete. The section primarily references ACI 318-08 [7] Chapter 21 on Earthquake-Resistant Structures and UFC 3-340-02 requirements. Sections 9.2.2.6 and 9.2.2.7 include requirements that depart from ACI 318-08 and UFC 3-340-02 for mechanical and welded splices, respectively.

Mechanical splices are limited to zones that remain elastic unless ASTM A706 reinforcing steel is used, which is a higher grade of steel used for reinforcing bars. Welded splices are also limited to zones that remain elastic unless it can be shown through performance qualification testing that the bar can develop the full tensile strength. Both of these splices do not require demonstration of ductility as required in UFC 3-340-02. Section 9.2.3 on columns references several portions of ACI 318-08 Section 21.6 on special moment frame members subjected to bending and axial load. This section is in agreement with the analysis and highlights the improved performance due to special moment frame detailing. For flexural members, Section 9.2.4 references ACI 318-08 Sections 21.5.2.1 and 21.5.2.2 which specify requirements for the amount of reinforcement provided. Section 9.2.6 on slabs also provides requirements for reinforcing of slabs to provide resistance to progressive collapse and rebound.

CONCLUSIONS

This case study explores the use of seismic detailing, including structural integrity reinforcement and full-capacity mechanical butt-splices, to increase blast resistance of buildings. The discussion above clearly demonstrates that changes to the design codes enacted prior to the Murrah building blast, but after the building was designed and constructed, would have provided significant improvement in building performance.

Public buildings are currently being designed to either prevent a terrorist attack with large standoff distances or to resist the blast load demand. Private buildings can also be designed for blast resistance and protection with only relatively small modifications to the building design. Prevalent approaches involve seismic detailing to provide energy dissipation and ductility in addition to integrity requirements to prevent progressive collapse.

Designing for the prevention of progressive collapse is essential to prevent catastrophic loss of life in the event of structural element failure. Cost analyses have shown that detailing with full-capacity mechanical butt-splices for the Murrah building transfer girder and other spandrel beams would have increased the cost by approximately 1/8 of 1% of the total cost of the building [8]. This cost difference is insignificant and would have reduced the likelihood of progressive collapse.

As seen above, a number of the requirements included in the ASCE/SEI standard are a result of the investigation of the Murrah building collapse. As engineers we must continuously learn from these types of failures in order to improve our structures and our design codes. We have a responsibility to the public to do better in the future to prevent catastrophic failures, even in the event of terrorist attacks.

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Learning From Failures in Concrete Design and Construction

Norbert Delatte

Synopsis: The study of failures can help teach students how to ensure the satisfactory performance of buildings and bridges. A number of failure case studies have been developed for use in courses teaching reinforced concrete design, as outlined in the book “Beyond Failure.” These include failures in the construction process and in formwork, such as the Willow Island Cooling Tower. Some of the shear design provisions in current codes can be traced back to two collapses of Air Force warehouses in the mid-1950s. Three building collapses in the 1970s and 1980s showed the importance of punching shear. There is also much to be learned from reinforced concrete building performance under extreme conditions, such as the terrorist attacks on the Oklahoma City Murrah Building and the Pentagon (9/11). Another classic case is the collapse of a major section of the Ronan Point apartment towers in the UK in 1968, illustrating the need to properly tie precast building elements together. The collapse of the Laval, Quebec concrete bridge abutments in Canada shows the importance of providing continuity of reinforcement. This case study also offers the opportunity to illustrate the application of strut and tie models to analysis of complex reinforced concrete structures.

Keywords: compressive strength, failure, formwork, progressive collapse, punching shear, strut-and-tie models

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INTRODUCTION

The study of failures, particularly in a design course, can help teach students how to ensure the satisfactory performance of buildings and bridges. A number of failure case studies have been developed for use in courses teaching reinforced concrete design, as outlined in chapter 5 of the book “Beyond Failure¹.” These include failures in the construction process and in formwork, such as the collapse of the Willow Island Cooling Tower. Some of the shear design provisions in the current ACI 318 Building Code² can be traced back to two collapses of Air Force warehouses in the mid-1950s. Three building collapses in the 1970s and 1980s showed the importance of punching shear. These were the 2000 Commonwealth Avenue collapse in Massachusetts, the Skyline Plaza/ Bailey's Crossroads collapse in Virginia, and the Harbour Cay Condominium collapse in Florida. The latter two were investigated by the National Bureau of Standards, NBS (now the National Institute of Standards and Technology, NIST). There is also much to be learned from reinforced concrete building performance under extreme conditions, such as the terrorist attacks on the Oklahoma City Murrah Building and the Pentagon (9/11). Another classic case is the collapse of a major section of the Ronan Point apartment towers in the UK in 1968, illustrating the need to properly tie precast building elements together. The relatively recent collapse of the Laval, Quebec concrete bridge abutments in Canada shows the importance of providing continuity of reinforcement. This case study also offers the opportunity to illustrate the application of strut and tie models to the analysis of complex reinforced concrete structures.

In the U.S., a typical first course in Design of Reinforced Concrete Structures (such as CVE 422 at Cleveland State University) focuses on basic theory and design of reinforced concrete structures according to the American Concrete Institute (ACI) 318-08 (ACI 2008) code². Often the course is taught using a textbook based on that standard, such as Wight and MacGregor (2009)³. In most cases, only about half of the textbook material is covered in an introductory course, although some universities offer a subsequent advanced course on topics such as torsion, slender columns, plastic design, yield line theory, and so forth.

USE OF CASES IN THE CLASSROOM

For the past two years the course at Cleveland State University has included a number of failure case studies selected to illustrate the behavior of reinforced concrete structures and members. In addition to the course text (Wight and MacGregor, 2009) and code (ACI 2008), the course included excerpts from Chapter 5 (Reinforced Concrete Structures) of the book *Beyond Failure* (Delatte, 2008)¹. The excerpt was provided to students for advance reading before a class devoted to presentation of a series of failure cases. The first exam in the course asked the question, “What specific failure mode are flat plate structures particularly vulnerable to? Name a building that collapsed due to this failure mode.” It has been found, in this course and other engineering courses, that a reasonable upper limit on cases is four or five per course. The cases are selected to illuminate the specific technical principles and structural behaviors that govern the design and performance of reinforced concrete structures.

Short supplemental videos are also posted in the course BlackBoard online system. For fall 2010, these were the Willow Island Cooling Tower collapse and the Schoharie Creek Bridge collapse, taken from the History Channel's Engineering Disasters series, numbers 6 and 9 respectively¹. The handouts of the slides from the PowerPoint presentations on the cases are posted in BlackBoard as well.

CONCRETE MATERIALS AND CONSTRUCTION

Although construction methods are not the focus of the CVE 422 course, the designer is also responsible for construction specifications, and an understanding of material behavior and construction practices is of considerable value.

One case study of this type is the collapse of the Willow Island Cooling Tower at Willow Island, West Virginia, on April 27, 1978. Because this collapse case study focuses on concrete strength and formwork practices, it is more of a construction case study and is found in Chapter 9 (Construction Materials) of *Beyond Failure*¹. This

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case study is not covered in the CVE 422, because the same principles can be addressed in the 2000 Commonwealth Avenue collapse and the Bailey's Crossroads/ Skyline Plaza collapse case studies, which are discussed in this paper.

SHEAR MECHANISMS AND FAILURES

Shear in reinforced concrete can be a complex subject for students to grasp. First of all, the flow of forces can be difficult to visualize. Second, while the need to use stirrup reinforcement to enhance shear strength is straightforward, the need to provide minimum stirrups when the calculated forces show that they aren't necessary is not so obvious.

The roof collapses of the Air Force Warehouses at Wilkins Air Force Depot in Shelby, Ohio, on August 17, 1955, and at Robins Air Force Base near Macon, Georgia, on September 5, 1956, are often used to show the need for minimum stirrup reinforcement (e.g. p. 234 of Wight and MacGregor, 2009)³. These collapses are generally attributed to combined load, shrinkage, and thermal effects. In each case, 400 foot (122 m) lengths of roof girders functioned as single units due to defective expansion joints (pp. 130 – 133, Delatte, 2008)¹.

Three case studies illustrate the collapse of buildings under construction due to punching shear and similar mechanisms. The first of the three was the January 25, 1971 collapse of the 2000 Commonwealth Avenue apartment building under construction in Boston. Four workers were killed in this collapse. The project was characterized by an almost total lack of construction control and inspection. The floor plan and the extent of collapse are shown in Fig. 1.

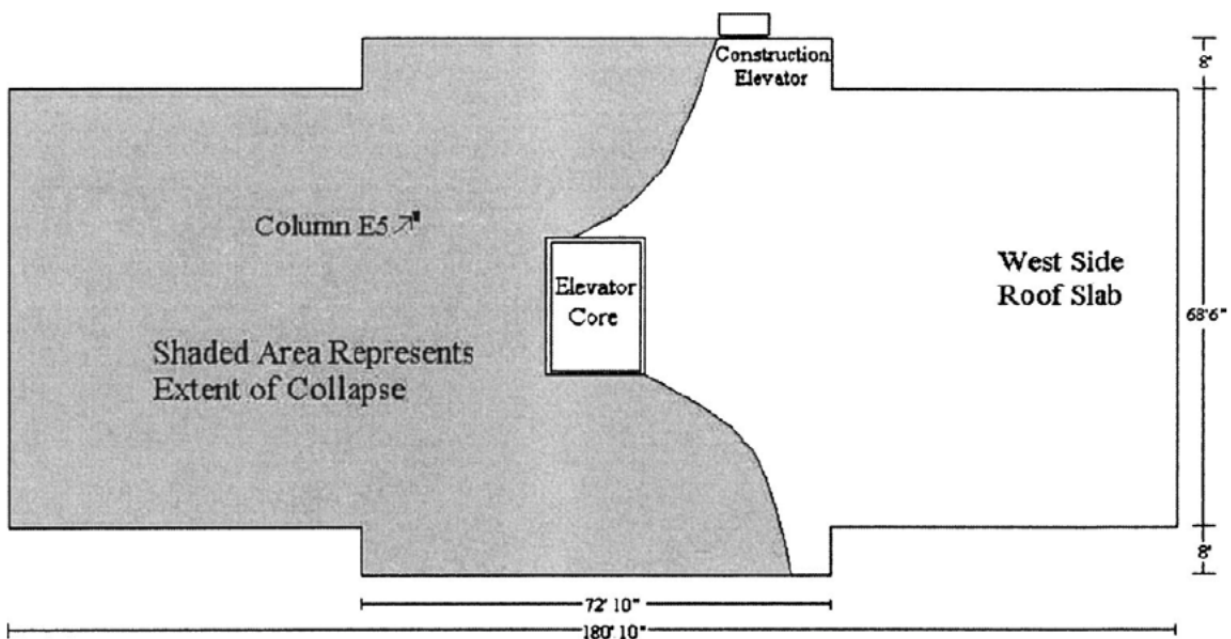


Figure 1 – Collapsed Section of 2000 Commonwealth Avenue^{1, 4}

This building used flat plate construction. A plate is a slab of uniform thickness supported on columns. This is a very economical type of construction for short spans and light loads because formwork is very simple and the absence of beams reduces the floor-to-floor height (p. 606, Wight and MacGregor, 2009)³. However, the problem with the system is that it is subject to two-way or punching shear of the slabs, which can be a brittle failure mechanism. Despite the importance of this failure mechanism, textbooks often suggest that the material not be covered in an introductory course but be deferred to an advanced course.

The punching shear mechanism is illustrated in Fig. 2. The slab breaks away and the column “punches” through the slab. The resistance to punching shear depends on the strength of the concrete, the effective depth of the slab, and the perimeter of the failure plane around the column. Clearly, the loads applied to the slab and the removal of shoring under the slab are also important. In the collapse at 2000 Commonwealth Avenue, the strength of the concrete was low and strength development was impaired by cold weather. Some of the reinforcement was improperly placed and poorly developed. The roof slab of the building held heavy mechanical equipment, and there was evidence that shoring had been removed prematurely. Details are provided in references 1 and 4. The City of Boston published a detailed report⁵.

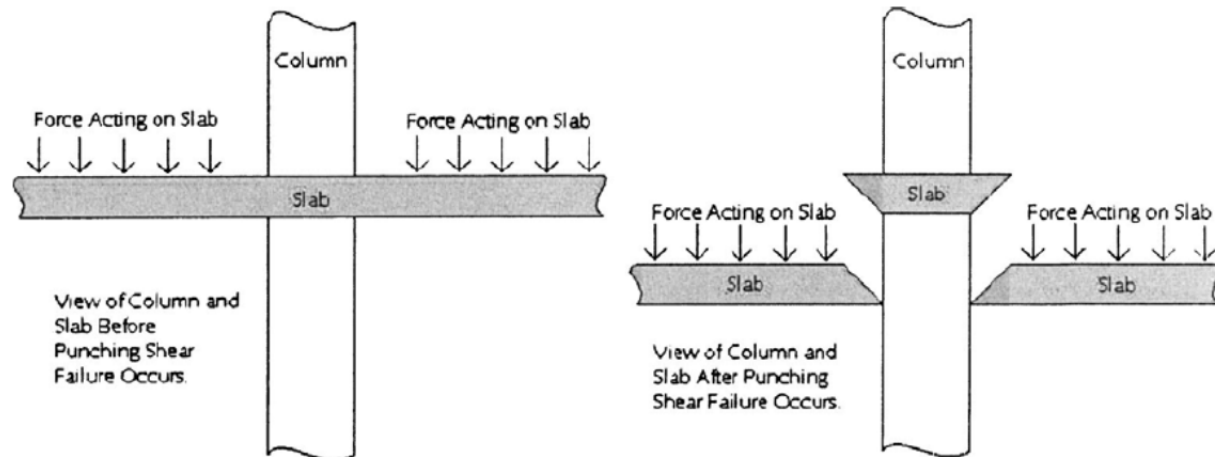


Figure 2 – Punching shear failure mechanism^{1, 4}

Only a few years after the 2000 Commonwealth Avenue collapse, a similar collapse occurred at Skyline Plaza, Bailey’s Crossroads, Virginia. On March 2, 1973, a section of the apartment building under construction collapsed to the ground. The collapsed structure is shown in Figs. 3 and 4. This collapse was investigated by the National Bureau of Standards (NBS) which is now the National Institute of Standards and Technology (NIST)^{6, 7}.

Once again, punching shear was the culprit. An entire section of the building collapsed to the ground, so that the structure looked like two separate buildings (Fig. 4). The rate of construction was approximately one floor per week, but in cold weather (e.g. March) rate of concrete strength gain was severely retarded. NBS used the maturity method to estimate that the compressive concrete strength was only about 1,000 – 1,500 psi (6.9 – 10.3 MPa) at the time of the collapse. The investigation also found that the collapse occurred while shoring was being removed several floors below a fresh concrete placement. The NBS published a full report on their investigation^{6, 7}.

Kaminetzky⁸ (p. 67) notes that the “collapse raised the issue of the extent of the engineer of record’s responsibility for the success and safety of formwork design and inspection. While the engineer’s contract specifically stated that he had no responsibility for field inspection, nevertheless a jury found him negligent. This was so because the pertinent code required that ‘a competent architect or engineer’ must provide supervision ‘where requested by the building official.’” Often, owners are not willing to pay engineers for the site visits.

Feld and Carper⁹ (p. 243) observed that a federal jury found both the architect and engineer negligent, and awarded large damages. The architect’s and engineer’s construction recommendations were not followed. The general contractor and building owner were not named, despite the fact that the failure was obviously due to mismanaged construction. They were shielded because the plaintiffs had already received awards under the Virginia Workers’ Compensation Act.



Figure 3 – Skyline Plaza/ Bailey's Crossroads Collapse (photo courtesy Nick Carino of NIST)



Figure 4 – Skyline Plaza/ Bailey's Crossroads Collapse (photo courtesy Nick Carino of NIST)

Delatte

The third punching shear case study was the collapse of the Harbour Cay Condominium on March 27, 1981 in Cocoa Beach, Florida. Eleven workers were killed and 23 were injured. The condominium was five stories tall, with shoring at all levels (Fig. 5). When the forms were removed, the structure collapsed completely with all floors pancaking to the ground (Fig. 6).



Figure 5 – Harbour Cay Condominium before Collapse (photo courtesy Nick Carino of NIST)



Figure 6 – Harbour Cay Condominium Collapse (photo courtesy Nick Carino of NIST)

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Figure 6 shows an important clue to the collapse. The column standing up shows a telltale sign of punching shear where the slab used to be attached to the column. This collapse was also investigated by the NBS^{10, 11}. The investigation found that the slabs were considerably thinner than the ACI minimum typically used for deflection control. Moreover, the reinforcing steel had been placed too low in the slab because chairs of the wrong height had been used, which made the effective depth of the slab too small. The column sizes were too small. In addition to causing the problem of column reinforcement congestion and poor consolidation of concrete, this meant that the perimeter around the columns to resist punching shear was too small. The concrete, however, had been of adequate strength.

This failure was due to design error. Incredibly, no punching shear calculation had been made for the concrete floor slabs. The chairs used to support the slab steel were $4\frac{1}{4}$ inches (108 mm) high, which coupled with the thin slabs led to a very small effective depth⁸. According to Feld and Carper⁹ (p. 274) the structural engineer was a retired NASA engineer who hired another retired NASA engineer to do the calculations.

PERFORMANCE UNDER EXTREME CONDITIONS

One early case study of concrete building performance under extreme conditions was the partial collapse of the Ronan Point apartment tower in London, U.K., on the morning of May 16, 1968. Ronan Point was 22 stories tall and was built using a system of precast panels fastened together. As the occupant of one apartment lit a stove to brew her morning tea, leaking gas exploded and blew out an exterior load-bearing wall. An entire corner of the building collapsed to the ground, killing four. Subsequent investigations showed that the connections between the panels were very poorly constructed (Fig. 7). A very light explosion was able to cause a disproportionate collapse of the structure. This case study led to changes in building codes to improve structural integrity and robustness¹². The U.K. government published a thorough investigative report¹³.

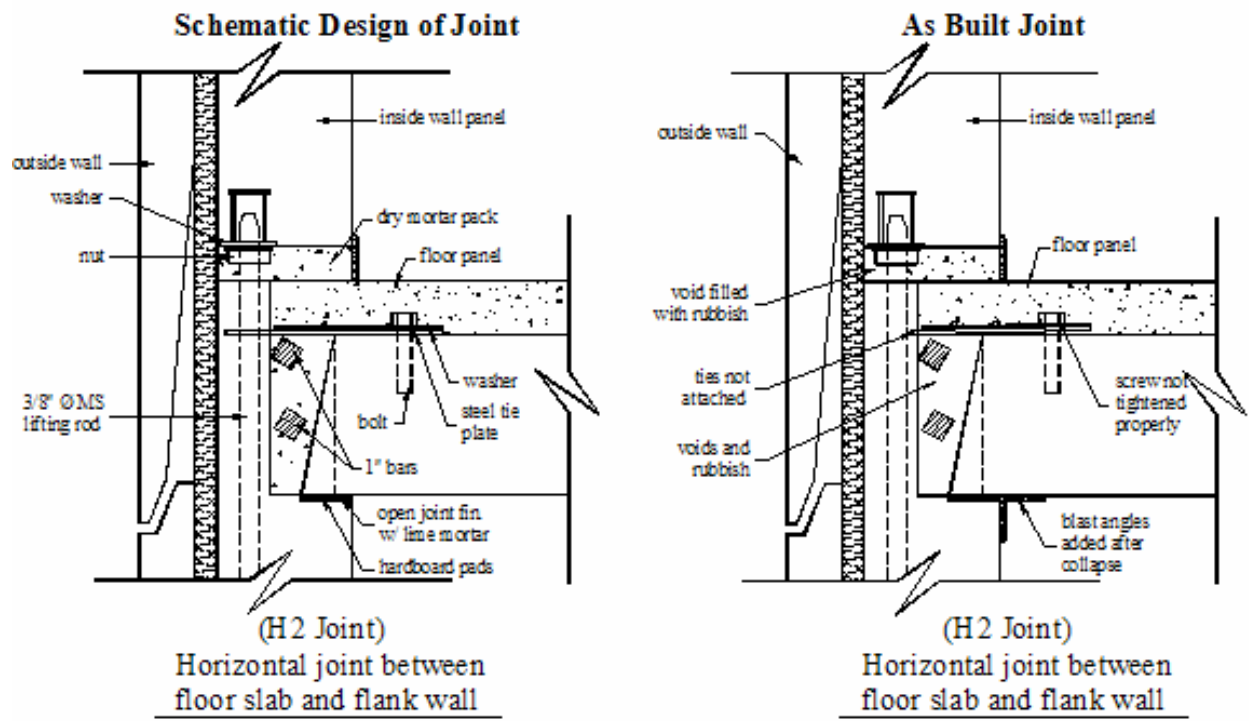


Figure 7 – Ronan Point connection^{1, 12}

In recent years, reinforced concrete structures have been tested under extreme events, including terrorist attacks. These include the bombing of the Oklahoma City Murrah Federal Building on April 19, 1995, and the

attack on the Pentagon on September 11, 2001. Although neither building was designed for this type of event, there are lessons to be learned by contrasting the performance of the two structures.

By pretty much any criterion the Murrah Federal Building performed poorly. Approximately half of the floor plan was destroyed and 169 people were killed. Once a critical column was shattered, the continuity of the structure was severely disrupted. Floor slabs were blasted upwards and shattered, and then fell downward and pancaked to the ground. A detailed analysis is provided by Hinman and Hammond¹⁴ (1997).

On the other hand, the performance of the Pentagon to the September 11, 2001 attack is generally considered to be very good. The building had been constructed during World War II, and the original design concept called for it to store records after the war, so it was designed for high floor live loads of 150 psf (7.2 kPa). Circular columns with spiral reinforcement were used, and beam bottom reinforcement was continuous through the columns. The building was also in the process of being renovated to improve safety against blast.

As a result, the collapse of the building was localized to the immediate area of the aircraft impact, and the damage did not spread disproportionately. Columns continued to carry loads even though severely bent. A comprehensive building performance report has been published¹⁵.

Obviously, earthquakes represent an important type of extreme condition, and the performance of reinforced concrete buildings in these events is of great importance. However, Cleveland is an area of relatively low seismic activity, so earthquake performance is not typically included in an undergraduate reinforced concrete design course. In other geographical areas more prone to earthquakes, however, earthquake case studies would obviously be very important.

CONTINUITY OF REINFORCEMENT AND STRUT-AND-TIE MODELS

Although collapses of structures in service without an immediate, obvious triggering cause are rare, one such case was the failure of the Autoroute 19 de la Concorde Overpass in Laval, Quebec, on September 30, 2006. This bridge collapse killed five people and injured six.

The bridge consisted of parallel prestressed concrete box beams resting on cast-in-place concrete cantilevers extending out from the abutments. The supports doubled as expansion joints, and were subjected to unpredictable tension forces from the expansion and contraction of the bridge.

The cantilever supports relied on a complex reinforcement detail to transfer bearing forces to the top bars of the cantilevers through stirrups. However, the tops of the stirrups were placed slightly below the top reinforcement, leaving a horizontal plane of weakness. The support peeled away along that plane of weakness.

This case study offers the opportunity to illustrate the use of strut-and-tie models in design and analysis of reinforced concrete structures. The provincial government of Quebec published a lengthy report, with versions in both French and English¹⁶.

STUDENT SURVEY RESULTS

The requirements that civil engineering programs have to meet now, and will have to meet in the future, are contained in a number of documents. These include the general and program specific ABET Inc. Engineering Accreditation Commission (ABET EAC) criteria¹⁷ and the Civil Engineering Body of Knowledge (BOK)¹⁸. The Civil Engineering Body of Knowledge (BOK) has been developed by ASCE over several years, and is expected to affect ABET EAC accreditation criteria at the bachelor's and master's level for civil engineering programs. Some of the BOK outcomes will present challenges for faculty and programs. It will be necessary to develop new educational materials and teaching techniques to facilitate BOK implementation. Some of the ABET outcomes listed in Table 1 tend to be hard to achieve with a "traditional" engineering curriculum, such as those that relate to ethics, life-long learning, and contemporary issues¹⁹.

For the fall 2009 offering of CVE 422, 15 students were surveyed about the use of case studies in the course. The survey was repeated in the fall 2010 offering of the same course with 14 students. Results are shown in tables 1 and 2. The Likert scale used ranged from 1 (strongly disagree) to 5 (strongly agree). The tables show the

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average value and standard deviation (SD) for the two years. Similar results have been reported previously for other courses²⁰. In Table 1, the students were surveyed about the contributions of case studies to meeting the program outcomes as related to accreditation requirements. In Table 2, the students were asked how the various elements of the course (textbook, lectures, homework assignments, exams, and case studies) contributed to their interest in and understanding of course material.

Table 1 – Student Survey Responses on Failure Case Studies related to ABET Outcomes (Fall 2009 and 2010)

The case studies contributed to:	2009		2010	
	Ave	SD	Ave	SD
my ability to apply knowledge of mathematics, science, and engineering;	4.13	0.64	4.07	0.73
my ability to design and conduct experiments, as well as to analyze and interpret data	3.60	0.83	3.71	0.73
my ability to design a system, component, or process to meet desired needs, using the principles of equilibrium;	3.67	0.82	3.86	0.77
my ability to function on multi-disciplinary teams	3.20	0.94	3.71	0.73
my ability to identify, formulate, and solve engineering problems;	4.20	0.68	4.36	0.50
my understanding of professional and ethical responsibility	4.73	0.46	4.64	0.50
my ability to communicate my problem solutions effectively;	3.47	1.06	3.86	0.77
the broad education necessary to understand the impact of engineering solutions in a global and social context	4.00	0.93	4.29	0.61
my recognition of the need for, and an ability to engage in life-long learning	4.13	0.92	4.43	0.65
my knowledge of contemporary issues	4.07	0.70	4.43	0.65
my ability to use the techniques, skills, and modern engineering tools necessary for engineering practice	3.60	1.06	4.00	0.78

It can be seen that the results are particularly strong for ability to identify, formulate, and solve engineering problems (4.20/4.36), understanding of professional and ethical responsibility (4.73/4.64), broad education to understand the impact of engineering solutions (4.13/4.43), and knowledge of contemporary issues (4.07/4.43).

Table 2 shows that the case studies had the strongest contribution (or almost strongest) to student interest in the course material (3.86/4.18), as well as a strong contribution to understanding of the course material (3.71/4.09).

Table 2 – Student Survey Responses on Relative Contributions (Fall 2009 and 2010)

How well did each of these elements contribute to your INTEREST in the course material?				
	2009		2010	
	Ave	SD	Ave	SD
Textbook	3.29	0.73	3.55	1.13
Lectures	3.93	0.73	4.00	0.78
Homework	3.64	0.93	4.00	0.77
Exams	3.29	0.83	3.36	1.03
Cases	3.86	0.86	4.18	0.87
How well did each of these elements contribute to your UNDERSTANDING of the course material?				
Textbook	3.79	0.70	3.91	0.83
Lectures	4.43	0.65	4.27	0.65
Homework	4.29	0.61	4.18	0.60
Exams	3.55	0.82	3.90	0.90
Cases	3.71	0.61	4.09	0.70

There really isn't any way to do a control group with this type of study, since you can't ask students who have not had failure case studies as part of the course if the case studies had any impact. The survey will be repeated whenever the course is offered, and the results will be compared between offerings of CVE 422 and to other civil engineering and engineering mechanics courses. Future work will also examine if the inclusion of case studies can improve engagement and performance of women and other underrepresented minorities.

SUMMARY AND CONCLUSIONS

The case for the inclusion of case studies in engineering courses, particularly design courses, has been made elsewhere – for example, in Chapter 1 of Reference 1. There are a number of case studies, as outlined in this paper that could be used in a reinforced concrete design course. More information on these cases and others may be found in the references cited below, in Reference 1 and in the failure case studies project web site <http://matdl.org/failurecases/>²¹.

Obviously, too much of a good thing can quickly become too much. The author uses five case studies in the CVE 422 Reinforced Concrete Design course. These are the Air Force Warehouses, 2000 Commonwealth Avenue, Skyline Plaza/ Bailey's Crossroads, Harbour Cay Condominium, and the Laval Quebec Bridge collapse.

The students responded well to the case studies, and survey results showed that they made important contributions to ABET outcomes. This can be particularly important for civil engineering programs struggling with the challenge of demonstrating that students meet the ABET outcomes that tend to be hard to achieve with a "traditional" engineering curriculum, such as those that relate to ethics, life-long learning, and contemporary issues. The contributions to student interest in and understanding of course material are also noteworthy.

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Using Historical Cylinder Strength Data for Structural Evaluation

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Synopsis: Section 20.2 of ACI 318-08 gives provisions for determining required dimensions and material properties for the strength evaluation of an existing concrete structure. Subsection 20.2.3 states “If required, concrete strength shall be based on results of cylinder tests from the original construction or tests of cores removed from the part of the structure where the strength is in question. For strength evaluation of an existing structure, cylinder or core test data shall be used to estimate an equivalent f_c' .” The commentary cites two methods presented in ACI 214.4R for determining an equivalent-to-specified f_c' using cores from an existing structure. There are no criteria provided or references cited, however, suggesting how to compute an equivalent f_c' based on cylinder strength data. The paper first identifies necessary conditions for using original cylinder test data to determine a concrete compressive strength for structural evaluation. To investigate strength compliance during initial construction, methods to compute an equivalent-to-specified f_c' value are presented that are based on inverting the conventional acceptance criteria given in ACI 318-08 Section 5.6.3.3. To evaluate an older structure, a method to determine an equivalent-to-specified strength based on the lower-bound fractile of the concrete strength represented by f_c' is presented.

Keywords: concrete cylinder tests, equivalent specified concrete strength, specified concrete compressive strength, strength evaluation, structural concrete

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INTRODUCTION

Section 20.2 of ACI 318-08 gives provisions for determining required dimensions and material properties for the strength evaluation of an existing concrete structure. Subsection 20.2.3 states “If required, concrete strength shall be based on results of cylinder tests from the original construction or tests of cores removed from the part of the structure where the strength is in question. For strength evaluation of an existing structure, cylinder or core test data shall be used to estimate an equivalent f'_c . The method for obtaining and testing the cores shall be in accordance with ASTM C42.” The commentary to this subsection draws the user’s attention to two methods developed by ACI Committee 214 for determining f'_c from cores taken from an existing structure. There are no criteria provided or references cited, however, concerning the computation of an equivalent f'_c based on cylinder strength data.

This paper is intended to rectify this deficiency. It starts with a brief discussion of potential pitfalls that can occur when using original cylinder test data for strength evaluation of an existing structure, and identifies conditions necessary to follow this procedure. Then, paralleling the structure of ACI 214.4R-10 (ACI 214, 2010), it offers two procedures: one applicable for investigating low cylinder strength test results in new construction, based on the provisions of ACI 318-08 (ACI 318, 2008) Section 5.6.3.3, and the other applicable for the strength evaluation of an existing structure based on determining a lower-bound estimate of the fractile of the concrete strength represented by f'_c as derived from the cylinder strength data. A brief comparison of the results obtained using these two methods is then presented and discussed.

RESEARCH SIGNIFICANCE

The provisions in Section 20.2 of ACI 318-08 permit the determination of the compressive strength of concrete in an existing structure using the original cylinder strength test results. Although this procedure may be flawed because it does not directly measure the in-place strength of concrete in a structure, it provides a low-cost approach that is clearly welcome in practice. Current design criteria, including particularly the calibration of strength reduction factors used to verify structural safety under ultimate loading conditions, recognize that the specified compressive strength of concrete is a lower bound on the cylinder strength. This paper therefore proposes two procedures for the computation of an equivalent f'_c based on cylinder strength data for two extremely common circumstances: (1) when low strength test results occur during construction of a structure; and, (2) when an equivalent-to-specified strength is sought for the strength evaluation of an existing structure after it has been occupied.

NECESSARY CONDITIONS TO USE CYLINDER STRENGTH DATA

The use of cylinder data to assess the strength of concrete in an existing structure is potentially deceiving because cylinders give no direct indication of actual in-place strengths. If the concrete was improperly cured or has deteriorated over time, the actual in-place strength could be markedly less than the strengths obtained by testing the original cylinders. Conversely the actual in-place strengths could be markedly greater than the original strengths, as cement continues to hydrate over time, particularly for coarse-ground cements produced before the 1970s. For example, 40-year old cores from the main foundations of Lions’ Gate Bridge in Vancouver indicated compressive strengths between 39 and 86 MPa, even though the original specified strength was believed to be 21 MPa (Buckland, 1981).