<u>Exhibit E</u>

Articles on Cinder Concrete

Cinder Concrete Slab Construction

By Ciro Cuono, P.E.

The great fires of the 1800s in Chicago, New York, and elsewhere spurred a technology race to develop the best fireproof floor system. The years between the 1870s and 1940s represented a golden age of new technology in structural systems. Cast iron, wrought iron, structural steel and reinforced concrete framing systems, terra cotta arch construction, cinder concrete slabs, and many proprietary systems were introduced during this period. Although now known as "archaic" structural systems, as they are no longer used and have been replaced with modern methods and materials, these systems represent a large portion of our building stock.

Of these varied archaic systems, cinder concrete slab construction became one of the most dominant structural slab systems used from the 1920s to the 1940s. This article explores the origin, history, design, performance and relevance today of cinder slab construction with focus primarily on use in New York City (NYC); however, it was used in many other parts of the country as well.

Cinder concrete slab construction, also known as cinder arches, "goulash" construction, or even "short span arch construction", was a type of reinforced concrete slab system consisting of low strength concrete which used cinders as an economic substitute for stone aggregate and draped wire mesh as reinforcement.

Unlike stone aggregate concrete with reinforcing bars, these systems were not really "reinforced concrete" in the conventional sense but actually tensile structures encased in a light weight low strength concrete. This subtle but key concept can be the source of misunderstanding in dealing with these systems. The steel draped wire mesh acted as a tensile catenary system which carried all loads in tension between steel beams. The cinder concrete provided a walking service, transferring loads to the tension wires and acted as fireproofing protection for the steel wires.

Although this type of system is no longer specified, it is very relevant to engineers and architects today, not only in NYC, but in other cities as well since many of our office buildings, residential buildings, school buildings, industrial buildings etc. are made with these types of floor systems. As a result, it is important to understand their origin, history, performance, strengths and weakness when planning renovations, and repairing defects and deterioration.

History and Origin

Cinder arch construction developed as a result of economic and social forces. As the concrete industry began to develop in the United States (US) in the late 1800s and early 1900s, the key ingredients took shape to form this new type of construction.

Welded wire mesh was first patented in 1901. Although it had a variety of uses, its use took off in early concrete road construction. The early wire mesh was triangular and woven, and then rectangular in shape. From road construction it began to enter the building market where rolls of wire mesh could be easily shipped and rolled out on a job site. The "cinder" part refers to cinder and clinker, by products of coal generating plants, recycled and used to replace more expensive aggregates. The NYC empirical tables referred to "clean boiler cinders" and Anthracite or coal cinders. This incidentally provided good fire resistance which was validated in various tests.

"Draped" mesh refers to wire mesh placed over the tops of steel floor beams and then draped down at the mid-span between the beams, thus creating the "catenary" or "hung chain" which provided optimal geometry for essentially a cable system in tension.

The high load capacity, excellent fire proofing properties, light weight, and ease of construction (rolling out a wire mesh versus laying out reinforcing bars), made these floor systems the primary choice for many engineers and builders. By the 1930s, they seem to have replaced terra cotta arch construction and many other proprietary systems.

It seems most of the testing and early uses in building construction occurred in NYC where many office and residential buildings built prior to World War II are still functioning quite well, the most famous of which is probably the Empire State Building.

Testing, Analysis, and Design

Many tests were conducted in NYC, over several years, as part of the technology race for fireproof floor systems.

One such test was conducted by Professor Ira Woolen at Columbia University in conjunction with the City Building Bureau in 1907 and 1908. The test consisted of a fire, water, and load test of a cinder concrete slab with 5-foot and 8-foot spans and reinforced with triangular wire mesh. The cinder concrete contained "boiler cinders". Specimens were load tested to a compressive strength of 1,000 (pounds per square inch (psi).

The results of the testing were good, withstanding a four-hour fire at approximately 1,700 degrees Fahrenheit and sustaining a 600 psf dead load.

Another significant test, in a series of many tests, was conducted in the summer of 1913 by Harold Perrine of Columbia University in Long Island City, NY. The test consisted of the construction of three types of floor systems; a cinder slab, a flat terra cotta arch, and a gypsum slab (also reinforced with welded wire mesh). The testing, funded by a

Ciro Cuono, P.E., is a Principal at Cuono Engineering PLLC, a structural engineering firm located in Port Chester, N.Y., and is an assistant adjunct professor of structural engineering at the Bernard and Anne Spitzer School of architecture at The City College of NY. He may be reached at ccuono@cuonoengineering.com.

Allowable load

The allowable load shall be determined by the following formula:

$w = 3CAs/L^2$

where: w = gross uniform load (psf)

As = cross sectional area of main reinforcement (sq. in. per ft. of slab width)

L = clear span between steel flanges in feet. (L shall not exceed ten feet in any case, and when the gross floor load exceeds two hundred psf shall not exceed eight feet)

C = the following coefficient for steel having an ultimate strength of at least fifty-five thousand psi;

1. For lightweight aggregate concrete:
   a. twenty thousand when reinforcement is continuous.
   b. fourteen thousand when reinforcement is hooked or attached to one or both supports.

(1) When the above formula is used the reinforcement shall be hooked or attached to one or both supports or be continuous.

(2) If steel of an ultimate strength in excess of fifty five thousand psi is used, the above coefficient C may be increased in the ratio of the ultimate strength to fifty five thousand but at most by thirty percent.

Figure 1. Excerpt from the 1968 New York City Building code (27-610) showing an empirical formula for cinder slab construction (carried over from earlier versions of the code).

Figure 3.

fireproofing company, was done to compare the fire resistivity of the three types of floor systems. Each was subjected to fire and test loading. The slabs were subjected to four hours of fire that was approximately 1,700 degrees Fahrenheit and then rapidly cooled with cold water, all the while carrying 150 pounds per square foot (psf) of pig iron.

After 24 hours of cooling, the slabs were loaded with further weight. The cinder slab had the best overall performance, with minimal damage from the fire and supporting 600 psf with only ½ inch deflection.

According to Frank Eugene Kidder, (a famous author of engineering handbooks in the early 1900s), some earlier tests conducted in 1902 had 4½-inch cinder slabs load tested to approximately 1,400 psf!

The successful testing and market use led to a codification of cinder floor slabs in NYC. The building code contained empirical formulas for determining slab thickness and wire mesh areas for many years (*Figure 1*).

These "empirical" formulas were essentially based on statics of a tensioned cable. The design became simply a matter of calculating a wire mesh area, or picking out the area from a load and span chart.

The cinder concrete itself was essentially unimportant. If conducting a modern compression core test on one of these slabs, a good result would be in the range of 700 psi – a result woefully unacceptable for a slab that is conventionally reinforced.

Construction

A typical cinder slab mix, often found on many old drawings, might be a 1:2:5 mix (1 part cement, 2 parts sand and 5 parts cinders) ranging in unit weight from 85 pounds per cubic foot (pcf) to 110 pcf. Touching a sample piece of cinder slab in the field feels like a piece of pumice stone. This light weight resulted in a material savings for the steel frames and foundations, making it very appealing as a floor slab system.

A typical slab was 4 inches to 5 inches thick, although 3½ inches thick can be found in many old buildings. Usually the top of the slab is at the beam elevation or just above it. The beams and slabs were then topped with a layer of loose cinder fill, which provided fireproofing to the top flanges. Within this fill layer were beveled wood sleepers, usually 16 inches on center. A hardwood floor could then be nailed to the sleepers. This fill layer was typically 2 inches to 2½ inches thick. At flat roofs, where pitch was required for drainage, the fill could be 6 inches to a 1 foot or more (*Figure 2*).

The wire mesh was draped, as mentioned above, and hooked around the flange of the end or perimeter beams.

The steel beams were encased in concrete for fireproofing. Typical spans ranged from 5 feet to 8 feet.

Performance

The performance of cinder slabs is rather amazing when one considers some of the inherent weaknesses of their design. The demonstrated analytical and historical strength of steel cables is well documented. As an essentially pure tensile structure, there seems to be a robust capacity for overloading. However, the small diameters of the cables or mesh result in a small robustness once there is the susceptibility to corrosion. Roof slabs and slabs near plumbing lines or below wet areas of construction (for example a restaurant kitchen floor) are

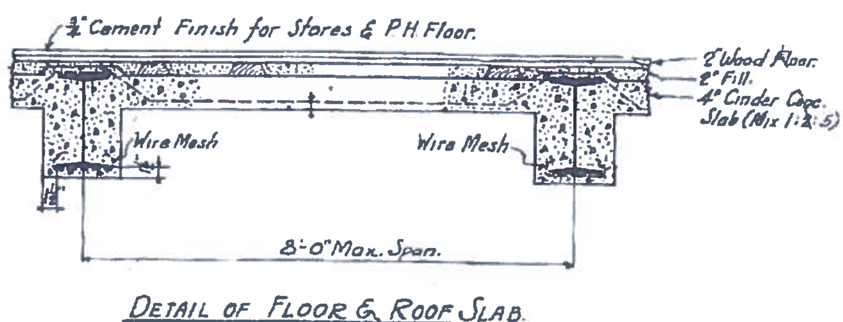

Figure 2.

common sources of leaks. The author has personally observed the underside of slabs that were subjected to long term corrosive environments, resulting in severely spalled slabs. From the floor one may observe the exposed mesh with an obvious rust color; however, upon close inspection one may find the wires severely corroded, snapped, or even completely disintegrated leaving behind a streak of rusting that almost looks like a partially corroded wire (*Figure 3*). One can only wonder how a condition like this has not resulted in a collapse. Perhaps a combination of redistributions at adjacent more fully intact areas, conservative loading requirements, friction, and other "ignorance factors" has prevented more disastrous results. To this author's knowledge, there is no significant documented major failure of these types of floor systems.

The ductility of steel mesh and the obvious signs of spalling have perhaps helped as well, as these signs of impending disaster usually signal a building owner to call in an engineer and provide some type of repair.

Figure 4.

Modern Issues

Since cinder concrete arches are no longer used, it would seem an "archaic" structure. In NYC, however, they are so ubiquitous that a working knowledge of their design and construction is a prerequisite to engaging in renovation work.

The usual issues have to do with either planned renovations, where loading changes and opening or closing of stair, mechanical or elevator shafts occur (*Figure 4*), or repairs due to rusting and corrosion.

Their long history of good use and tremendous load capacity from testing generally makes analysis fairly easy. Armed with a tape measure and a caliper, an engineer can take a few spot field measurements of the wire size and spacing and, in conjunction with the empirical formulas from decades ago, quickly arrive at a safe loading capacity.

Reframing openings can be tricky, since loss of anchorage or continuity of the mesh could theoretically relax the mesh. Many engineers often require contractors to tack weld any exposed mesh to the steel beams, especially adjacent to newly cut slabs.

Repairs are more complicated. Cinder concrete is extremely porous and lightweight. Water from leaks, from old steam lines, or roofs and parapets gets absorbed by the cinder concrete and can stay there for years, slowly corroding the wire mesh. The combination of the cinder aggregates and water can react to create sulfuric acid which, along with poor resistivity of the cinder concrete, can lead to severe corrosion.

The expansion from corroding wire mesh can crack and spall the underside of cinder slabs. Often a small spall is noticed and upon a few "whacks" of a sounding hammer, the entire underside can quickly spall off leaving the rusting wire mesh completely exposed. Caliper measurements can be used to recalculate a remaining capacity, assuming further corrosion is arrested. However, this can be impractical since conditions can vary greatly even in a few bays; thus, a few spot measurements may not give a reliable result.

An overhead repair mortar could be applied to patch the underside of a spalled slab; however, this cosmetic repair will not restore any lost capacity. New low profile steel beams (such as channels, angles, or tubes) can be installed below a defunct slab to reduce the span in lieu of a total demolition and replacement.

On a roof, where the loose fill may be quite thick, this fill can be removed and replaced with a new modern reinforced concrete slab spanning between the tops of the existing steel beams, thus abandoning the old slab in place and using it as form work only.

The creative engineer can find ways of working around a deteriorated slab. Understanding the limits of cinder slab construction is important to this process.

Another issue in modern renovations is hanging ceilings and mechanical units. Cinder concrete is notoriously unreliable with epoxy and mechanical anchors in tension. The original ceilings were often hung with wire that was hooked into an exposed portion of the slab wire mesh. Regular spots of chipped out concrete, exposing the wire mesh, can provide opportunities for easy field measurements. Load testing of anchors for light loads like a gypsum ceiling (say for 4 to 5 times the load) can be used; however, conditions could vary over short distances, making this method somewhat unreliable. The more conservative approach is to hang off the original steel beams, especially for anything heavier than a ceiling.

Renovation and Repair Examples

One example of a renovation of cinder slabs that has been successful is to take advantage of the loose cinder fill atop the structural slab to gain valuable space for new structure. As mentioned, the fill layer on roof slabs (of apartment buildings with flat roofs) was often quite thick; six inches to twelve inches was not uncommon. The removal of 10 inches of loose cinder fill is equivalent to almost 50 pounds per square foot (psf) of dead load. Removal of this dead load could be used to justify new additional dead and live loads, such as pavers for a roof deck. This "load balancing method" is quite convenient, especially if analysis of the existing framing cannot be done due to lack of original drawings and the inability to make destructive probes of the framing. Pitfalls to this method include the lack of an actual engineering analysis (what if the original framing was undersized?) or overestimation of the actual weight as the loose fill could be lighter than historic load tables may indicate. Also, consideration has to be given to fireproofing, as the top flanges of the steel beams were often fireproofed by the loose cinder.

A repair example, also at a roof slab, involved the removal of the loose fill to create a newer stronger conventional reinforced concrete slab that spans between the new beams. This is a convenient methodology where the existing slab is deteriorated. Rather than complete demolition and replacement (which could be more costly, and expose the interior to increased risk from temporary instabilities and the elements), the loose fill could be removed and then a new slab poured atop a thin layer of rigid insulation (to prevent bonding) (*Figure 5*). In an extreme case, where the existing slab was severely corroded, steel plates could be hung from the new slab to "lock-in" the old slab or prevent localized pieces from falling onto the occupants below.

In summary, the dominance of cinder slab systems from the 1920s to the 1940s and their continued successful performance in so many buildings today, despite some pitfalls that have been mostly related to corrosion issues, is a testament to their strength and versatility.▪

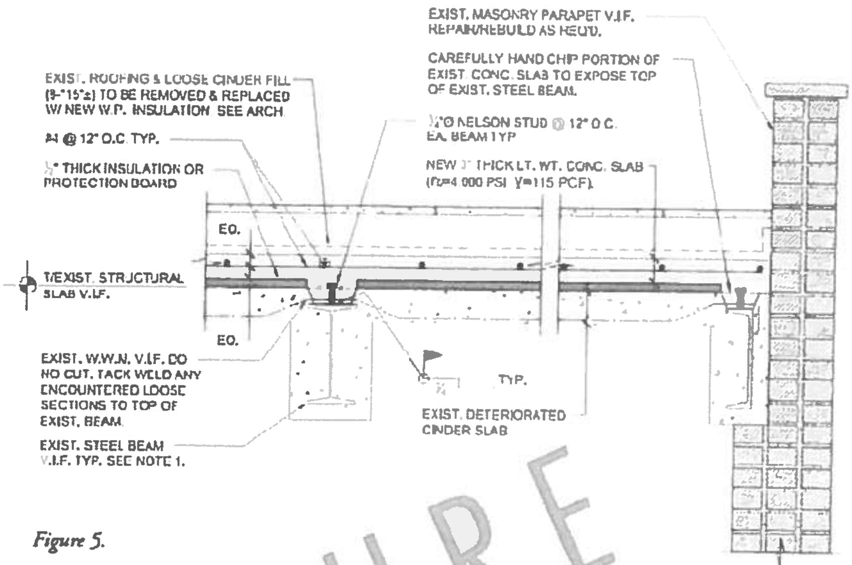

Figure 5.

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REPAIR OF DRAPED MESH CONCRETE SLABS

BY KIRK M. STAUFFER AND KEVIN C. POULIN

Draped mesh concrete slabs (supported by hot-rolled steel beams and girders) were one of the most prevalent one-way floor slab systems used in buildings constructed from the 1920s to the 1960s. These slabs consist of draped, welded wire reinforcement embedded in cinder-concrete (occasionally stone-concrete) that span between closely spaced steel beams. These slabs are not analyzed using current (modern) reinforced concrete design methods. As a result, modifications and repairs to these slabs require special knowledge of both their structural behavior and material properties.

Fig. 1: Typical method of mesh anchorage where welded wire reinforcement is bent and hooked around beam flange

Fig. 2: Perimeter of repair where existing reinforcement extends uninterrupted through repair area

HISTORY

Structural steel design had become fairly developed by the late 1880s, but reinforced concrete design was still evolving at that time. The disparity between the stages of development of the two materials became particularly acute when considering a floor system for newly developed mid-rise and high-rise steel-framed buildings. Typical early floor systems that spanned between steel beams were thick, heavy terra-cotta or brick arches covered with a relatively thin concrete topping and occasionally a layer of lighter-weight cinder fill sandwiched between the arch and the topping. These traditional, true-arch approaches slowly gave way to thinner and lighter proprietary reinforced concrete systems that acted structurally in catenary or beam action. As building heights increased (with greater knowledge and confidence in steel design), more developed concrete flooring systems became necessary.

During the 1920s through the 1960s, draped mesh cinder-concrete slabs were common because they used a lightweight, controlled-strength concrete that incorporated cinders, a readily available waste product of coal combustion. These slabs occur throughout New York City and other older urban areas where coal was burned throughout the city to provide heat and electricity. The Empire State Building, Chrysler Building, and Rockefeller Center are just a few of the many iconic buildings in Manhattan that use draped mesh cinder-concrete slabs. In the mid-1960s, however, as coal burning within cities was phased out and metal decking was becoming accepted, normalweight or lightweight concrete-on-composite-metal-deck slabs began to replace draped mesh cinder-concrete slabs as the preferred method for steel high-rise floor framing.

STRUCTURAL BEHAVIOR

Draped mesh slabs differ from modern reinforced concrete slabs because the steel welded wire reinforcement acts in one-way catenary action rather than in flexural action. At the high point in the slab, the welded wire reinforcement rests atop the steel beams (which are upset into the slab), then drapes down to a low point near the bottom of the slab at midspan. The welded wire reinforcement is placed continuously across several spans, often across the entire floor plate, and acts only in tension (behaving primarily like the main cables of a suspension bridge). To achieve the necessary structural capacity, the welded wire reinforcement needs to be continuous across all beams and anchored at the end spans (Fig. 1 and 2). As a result, any damage, deterioration, or intentional modifications to the slab must consider a means to maintain the continuity of the welded wire reinforcement, or provide alternative anchorage and/or supplemental supports. The concrete encases and bonds to the welded wire reinforcement and acts as a controlled-strength fill to transmit loads to the reinforcement. Because the concrete does not need to act fully as a structural material, its required strength is typically very low. Therefore, due to the availability of cinders to use as aggregate, cinder-concrete became the most common material used in these floor systems.

In 1896, the New York City Department of Buildings and Columbia University began a program to qualify and standardize some of the most common flooring systems. At that time, reinforced concrete design included numerous competing proprietary systems. They tested various floor systems for their load-carrying capacity and fire resistance. Specifically, tests for draped mesh cinder-concrete floor slabs took place at Columbia University between 1913 and 1914. These tests culminated in empirical formulae that were incorporated into the 1916 edition of the New York City Building Code (NYCBC). An example of the 1916 formulae for the design of concrete slabs in new construction is summarized as follows:

$$W = \frac{3CA_s}{L^2}$$

Cinder or stone concrete load-carrying capacity where W is the total load, in lb/ft^2; A_s is the area of steel, in in.^{2}/ft; L is the clear span, in ft; and C is the coefficient prescribed by the Code (varies for reinforcement, concrete type, and anchorage).

These formulae were included through the 1968 version of the NYCBC (which was in use until about 2008).

MATERIAL PROPERTIES

In addition to load-carrying capacity and fire resistance, Columbia University also evaluated the corrosion protection provided by the cinder-concrete. Their tests concluded that corrosion of the embedded welded wire reinforcement was prevented, as long as it was thoroughly coated with the cement mortar portion of the concrete. Despite the tests, other sources from the period questioned whether the acidic cinders would reduce the passive protection that concrete typically provides (through its inherently high pH) against corrosion of steel reinforcement. The literature of that time concluded that if the cinders were of good quality (typically completely combusted anthracite coal) and the concrete was proportioned and mixed properly, the cement paste should continue to passivate the corrosion reaction. The sources indicated that corrosion-related issues may exist, but they would likely be a result of the quality of the materials, mixing, or workmanship, and not with the composition of cinder concrete in general.

REPAIR METHODOLOGIES

Draped mesh cinder-concrete slabs have been repaired or strengthened due to deterioration (corrosion of the reinforcement and deterioration of the concrete resulting from exposure to water/moisture over time) (Fig. 3 and 4); due to damage (physical

Fig. 3: Deterioration of slab due to corroded reinforcement at slab underside

Fig. 4: Close-up view of corroded reinforcement at slab underside

Fig. 5: Partial-depth and full-depth slab damage from finish removal during a tenant fit-out

Fig. 6: Existing poorly executed repair made at a full-depth penetration

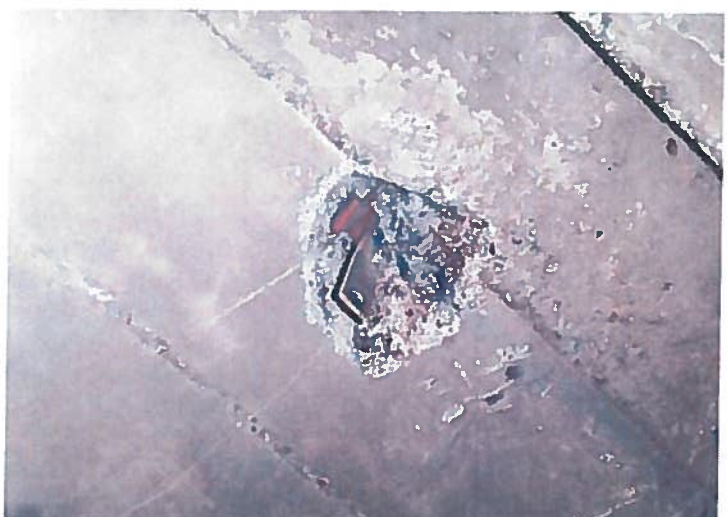

Fig. 7: Partial-depth slab damage where a wire of the reinforcement was exposed and used for supporting a hanger

damage revealed during tenant fit-outs) (Fig. 5 through 7); and for change of use (alterations due to change in occupancy or change in loading). Several typical and often-encountered repair methodologies are discussed in the following sections .

Note that this discussion of repairs to one-way draped mesh slabs will employ the terms "end span" and "adjacent span." In this context, an end span is the last span of a repair area that is directly adjacent to an existing, unmodified draped-mesh span. An adjacent span refers to an existing, unmodified draped mesh span that is adjacent to a repaired span.

PARTIAL-DEPTH REPAIR

Partial-depth repairs are the simplest type of draped mesh slab repairs and are commonly required when the reinforcement corrodes near its low point (at the middle of the span) or from shallow concrete damage that occurs during tenant fit-out renovations. Partial-depth repairs are relatively simple as long as the reinforcement has limited section loss, it remains anchored, and the required depth of repair into the slab cross section is limited. This type of approach usually involves repairing a small area, sometimes less than 3 in. (75 mm) diameter. Yet, in some instances, these repairs can include replacing the bottom cover of an entire span or infilling an area at the slab topside (Fig. 8). Other special instances of partial-depth repair can involve forming and pumping repair material in a localized area of deterioration at the underside of finished floors in occupied space.

Partial-depth repairs typically start with carefully removing the compromised cinder concrete, using hand methods, to expose the welded wire reinforcement and prepare the substrate. Power tools are not recommended for this work because of the brittle and unpredictable nature of cinder concrete. Furthermore, care should be taken during selective demolition to avoid loosening the reinforcement from the underside of the cinder concrete, and to avoid disturbing the bearing of cinder concrete onto the reinforcement. In every case, exposed reinforcement should be cleaned, examined for section loss, and coated with a corrosion inhibitor. In smaller areas where an individual wire of the reinforcement is exposed locally, the most common approach is to repair the area with a suitable trowel-applied overhead repair material. At locations where the corrosion of the reinforcement is widespread but the remaining cross section is still sufficient and the bearing of the cinder concrete on the reinforcement is not compromised, a new layer of expanded metal lath can be mechanically anchored to the underside of the slab and a new layer of repair material (troweled, shot, or formed and pumped), for fire and corrosion protection, can be applied. At locations where the cross section of the reinforcement is significantly reduced, full-depth, topping slab, or supplemental support repairs are required.

TOPPING SLAB REPAIR

Topping slab repairs can either be bonded (act compositely with the existing draped mesh cinder-concrete slab) or unbonded (where an independently supported slab is installed over the existing slab). Whenever a topping slab is added, or other significant changes are made to the permanent dead loads, the steel framing supporting the slabs should be evaluated.

For bonded topping slab repairs, the existing slab remains intact, and the continuity and anchoring of the draped reinforcement is usually not affected. Bonded topping slab repairs can supplement the strength of the existing slab for additional loading, or can be used to supplement the stiffness of the existing slab for new finishes that require low deflection (such as large floor tiles). To start a topping slab repair, the existing finishes, previous toppings, cinder fill, or other layers placed over the existing slab are removed. This is often beneficial because it removes the excess dead load from the system, and may achieve the desired ceiling heights. Next, the top surface of the cinder concrete of the existing slab is prepared to act as a substrate for the new repair material. Other than surface preparation, there is very little selective demolition of the existing slab, and exposed reinforcement is typically treated in a manner similar to a partial-depth repair.

Because bonded topping slab repair relies on composite action between the topping and the existing slab, and due to the low strength of the cinder-concrete slabs, direct-tension pulloff tests should be performed after the topping slabs have achieved sufficient strength. The results of these direct tension pulloff tests that evaluate the bond strength between the existing slab (substrate) and new topping slab can be used to correlate the available shear strength at the bond line between the topping and existing slabs. Without a proper bond, the needed composite action cannot be achieved.

Unbonded topping slab repairs can be installed over a damaged existing slab but are not always feasible if an increase of overall strength of the system, including the steel framing, is needed. The unbonded topping slab acts as an independent structural member that either replaces or upgrades the capacity of the existing slab and bears on the existing steel beams through the existing cinder-concrete slab. The existing slab is used only as a permanent formwork, and it is not relied upon for strength (other than the bearing stresses from the topping slab at the beams). If the cinder-concrete is in good condition but the welded wire reinforcement is severely corroded, the existing slab can remain in place below the new topping slab, provided that it can support its self-weight unreinforced. Less-deteriorated slabs can typically remain in place. In most instances, existing finishes, previous toppings, and cinder fill are removed for an unbonded topping slab. If the thickness of cinder fill is greater than the thickness of the topping slab, layers of rigid polystyrene can be installed to act as lightweight filler and to achieve the desirable finished-floor elevation (Fig. 9).

Topping slab repairs typically incorporate additional reinforcement. Unbonded topping slab repairs must be reinforced to provide the required flexural strength. However, it is possible for bonded topping slab repairs to be reinforced for shrinkage and crack control only. After installing reinforcement, either prepackaged repair materials (typically extended with aggregate) or ready mixed concrete can be used to place the topping slab.

Fig. 8: Selectively demolished partial-depth repair area at top of slab

Fig. 9: Unbonded topping slab repair area (with foam filler) adjacent to full-depth repair area on composite metal deck

FULL-DEPTH REPAIR

Full-depth repairs are required when the existing slab is severely deteriorated and can no longer support the required loading. To maintain the catenary action, the existing draped mesh must either be continuous, without modification throughout the repair span, or be anchored to the steel beams at the adjacent spans (Fig. 10). Usually, the welded wire reinforcement in the repair span is highly corroded, and continuity through the repair span is not possible. Thus, welding of the wire reinforcement to the steel beams at the edge of the repair span may be the only option, and new reinforcing is required for the design of the full-depth repair.

The procedure for a full-depth repair often starts with installation of temporary shoring in adjacent bays, placed as a precaution until the wire reinforcement can be properly anchored. Additional shoring, placed in the bays designated for replacement, is used as a work platform for demolition and to assist with formwork installation. Demolition proceeds with a saw cut at the repair perimeter, typically located over the centerlines of beams and girders. The depth of saw cut should not cut or damage the welded wire reinforcement and/or beam flange. Initially, demolition with small electric rotary or pneumatic hammers can remove the bulk of the cinder concrete at the center of the slab. Finally, selective demolition with hand tools is performed to remove the remaining cinder concrete within about one foot of the saw-cut perimeter. The extent of demolition and condition of the remaining cinder concrete, beams, and reinforcement are reviewed, and further action taken if necessary, prior to proceeding with installation of the new slab reinforcement. Once the new reinforcement is installed, ready mixed concrete or prepackaged repair material (typically extended with aggregate) is placed.

Fig. 10: Full-depth repair area with existing reinforcement and new additional reinforcement placed on wood formwork

Fig. 11: Closely spaced channels of supplemental support repair (shown after spray-on fireproofing was applied)

SUPPLEMENTAL SUPPORT REPAIR

At locations where a full-depth repair is performed but the reinforcement in an adjacent span cannot be properly anchored, it may be necessary to support the adjacent spans by providing supplemental supports (Fig. 11). In situations where the reinforcement's ability to carry load in tension through catenary action is interrupted due to lack of anchorage, closely spaced steel channels can be installed at the underside of the slab, parallel to its span, and connected to the existing steel beams. These new channels are sized and spaced to carry the self-weight of the slab and any superimposed loads. The slab can then be analyzed to span as plain concrete perpendicular to the new channels (parallel to the beams) using conservative material strength assumptions. In some cases, steel plates can be installed directly below that slab to act as a deck between the new channels. After the new steel is installed, the space between the underside of the slab and the top of the new steel is pumped or drypacked with a nonshrink grout.

CONCLUSIONS

When working in major cities with steel-framed buildings built in the first half of the 20th century (1920s through 1960s), it is important to understand and consider the type of concrete floor system (and concrete material), as well as its limitations and available remedial options, before embarking on implementation of any repair or strengthening work. If the existing floor slab is a draped mesh system, it is important to work with a structural engineer and a contractor who are experienced with this type of floor system. Remedial options are often limited and are governed by the behavior and material characteristics of the system. Even the smallest modifications (such as removing a small portion of a span for a new opening), if not planned and executed properly, can compromise the structural integrity of the floor over many bays. Providing adequate protection or shoring, and avoiding additional, unintentional damage, are equally important during the repair of these fragile systems. Proper experience and knowledge of cinder-concrete material characteristics and draped mesh concrete slab design are paramount to the repair of these floor systems.

***Kirk M. Stauffer**, PE (NY), SE (IL), is a Senior Staff I-Structures in the New York office of Simpson Gumpertz & Heger Inc. (SGH). He received his bachelor's and master's degrees in architectural engineering from Pennsylvania State University. His work commonly involves the assessment and rehabilitation of concrete structures, a wide range of concrete material science projects, and the interaction of concrete with other bonded materials ranging from waterproofing coatings to flooring. Stauffer is a National Ready Mixed Concrete Association Certified Concrete Technologist, the second Vice President of ICRI's Metro New York Chapter, is active with the Concrete Industry Board of New York, is a member of the American Concrete Institute (ACI), and is a member of ACI Committee 228, Nondestructive Testing of Concrete.*

***Kevin C. Poulin**, PhD, PE, is a Principal in the New York office of Simpson Gumpertz & Heger Inc. (SGH). He has a great depth of experience gained from over 20 years as a structural designer. His projects include renovation of existing buildings, design of new buildings, structural peer reviews, condition assessments, and feasibility studies. He is passionate about the restoration and adaptive reuse of existing buildings as a means to preserve the rich architectural history of New York City. His signature work includes numerous New York City landmarks. Poulin received his master's degree in structural engineering and his PhD in engineering mechanics from Columbia University. He is a licensed professional engineer in seven states, including New York, New Jersey, Connecticut, and Pennsylvania. He is a Past President of the Structural Engineers Association of New York.*