

SP-285 — 1

Concrete Mix Design: Key to Excellence in Performance of New Concrete Structures

By Jose M. Izquierdo-Encarnacion

Synopsis: In honor of Dov Kaminetzky, two case stories are discussed; one to reflect his passion for the correct usage and design for concrete and the other his other great passion, the forensic and technical evaluation of existing structures.

Keywords: Concrete Mix-Design; failure; forensic; quality control; ACI318 Code; investigation

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TOURISM PIERS#4 - OLD SAN JUAN, PUERTO RICO

Pier #4 was constructed in the 80's. There have been several piers constructed in the city during the twentieth century. Pier #4 was built to handle the operation of a new class of big cruise ships for the Caribbean Sea and was structurally designed to resist the docking forces of a 76,000 ton ship (nearly as heavy as the Navy ship, the Nimitz). Compared to modern cruise ships of 180,000 tons, this is now a comparatively small displacement tonnage.

Several new technologies were used for the design of Pier#4, but in particular was a new bumper system that dramatically absorbed up to 90% of the docking forces. The architectural designs for the roof of the building consisted of twenty four (24) concrete shells, each 94 ft (28.66m) in span and 6 in (150mm) thick. This building, compared to those used for modern cruise ships now looks quite small.



Figure 1 - East view of Pier#4

Back to the early 80's in Puerto Rico, there was a problem with concrete construction of the marine piers. The piers would promptly corrode reducing the life cycle of the infrastructure. Dov Kaminetzky enjoyed the networking that ACI provided, and through the Institute he provided a great wealth of knowledge to many professionals. For this project many professionals helped develop the proper criteria for the final material and structural design. In addition to ACI documents and publications, the best design manual used was from NAVFAC.

In terms of cement, Puerto Rico had a bigger problem because only a single Type-I cement was available on the island because of the local cement producers' control. Although other types of cement were available in the USA, they were not economically available in Puerto Rico due to transportation costs. It was decided to use a modified Type I cement. Although the local cement producer could produce Type II

Concrete Mix Design: Key to Excellence in Performance of New Concrete Structures

cement, the manufacturer did not want to disclose that they were actually producing Type II cement. Cement chemical composition affects the behavior of concrete. The most important component in cement for reducing reinforcement corrosion is the C_3A content. according to Verbeck 1968. The higher the C_3A , the more chloride can be bound as chloro-aluminate complexes—and thereby be unavailable for catalysis of the corrosion process. Where chloride is encountered along with sulphates in soil or groundwater, ordinary portland cement with a C_3A content from 5 to 8 per cent is desirable to be used in concrete rather than sulfate resisting cement. Figure 2 describes the effects of cement composition on durability properties of concrete.

Cement	Property	Cement Effects
	Placeability	Cement amount, fineness, setting characteristics
	Strength	Cement composition (C_3S , C_2S and C_3A), loss on ignition, fineness
	Drying Shrinkage	SO_3 content, cement composition
	Permeability	Cement composition, fineness
	Resistance to sulfate	C_3A content
	Alkali Silica Reactivity	Alkali content
	Corrosion of embedded steel	Cement Composition (esp. C_3A content)
Figure 2 – Effects of chemical components in Cement Properties		

During the 60's and 70's one of the largest problems in the U.S. and several other countries was a by-product from the use of admixtures. With the explosion of their use, concrete mixes started yielding higher compressive strength which then allowed the use of higher levels of water cement ratio. For example, concrete mixes of 3,000 psi compressive strength were produced with up to 0.80 water cement ratios. This sort of design yielded a concrete with high permeability and very low durability. As a consequence, engineers started specifying minimum water cement ratio. ACI 318 Code, Table 4.2.2 details requirements for special exposure conditions (Figure 3).

TABLE 4.2.2—REQUIREMENTS FOR SPECIAL EXPOSURE CONDITIONS

Exposure condition	Maximum water-cementitious materials ratio*, by weight, normalweight aggregate concrete	Minimum f'_c , normal-weight and light-weight aggregate concrete, psi
Concrete intended to have low permeability when exposed to water	0.50	4000
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45	4500
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources.	0.40	5000

* When both Table 4.3.1 and Table 4.2.2 are considered, the lowest applicable maximum w/cm and highest applicable minimum f'_c shall be used.

Figure 3 - ACI 318 requirements for special exposure conditions

Corrosion inhibitors had started to be available at that time such as DCI from W.R. Grace. In terms of the concrete mix design, the specifications were:

- a modified cement with C_3A less than 8%,
- a compressive strength of 5,000 psi,
- DCI as corrosion inhibitors and
- a 0.4 water cement ratio.

In addition, the US Army Corp-of- Engineers had started to extensively use epoxy coated reinforcement bars in their projects. Therefore, there was a big push from the Port Authority to use epoxy coated reinforcement. In terms of project cost epoxy coated reinforcement is expensive compared to the better alternative of producing a concrete with low permeability. Finally, the Port Authority was convinced to use the epoxy bars only selectively on the stirrups and the sidebars.

During the construction additional quality and assurance programs were established consisting of:

1. Regular testing of the cement used by the Ready Mix producer to verify the C_3A content.
2. Modulus of elasticity of concrete for removing the roof shells formwork at 24 hours.

Carefully planning the project requirements, combined with the evaluation of local materials with a cost effective solution yielded a structure that has been in use for over 25 years without corrosion.

PCC PAVEMENT RESTORATION PR-22, PUERTO RICO

PR-22 is a 51-mile (83-km) long toll road in the north coast of Puerto Rico that connects the cities of San Juan and Hatillo. The road is also known as the José de Diego Expressway. It is a 4-lane road for much of its length, but expands to up to 12 lanes in the San Juan metro area. The road is frequently congested, particularly during rush hour due to heavy commuter traffic; handling an average of 140,000 vehicles per/day.

Over the years the pavement deteriorated in some areas. Puerto Rico Highway and Transportation Authority requested bids for the repair of several segments as shown in Figure 4.

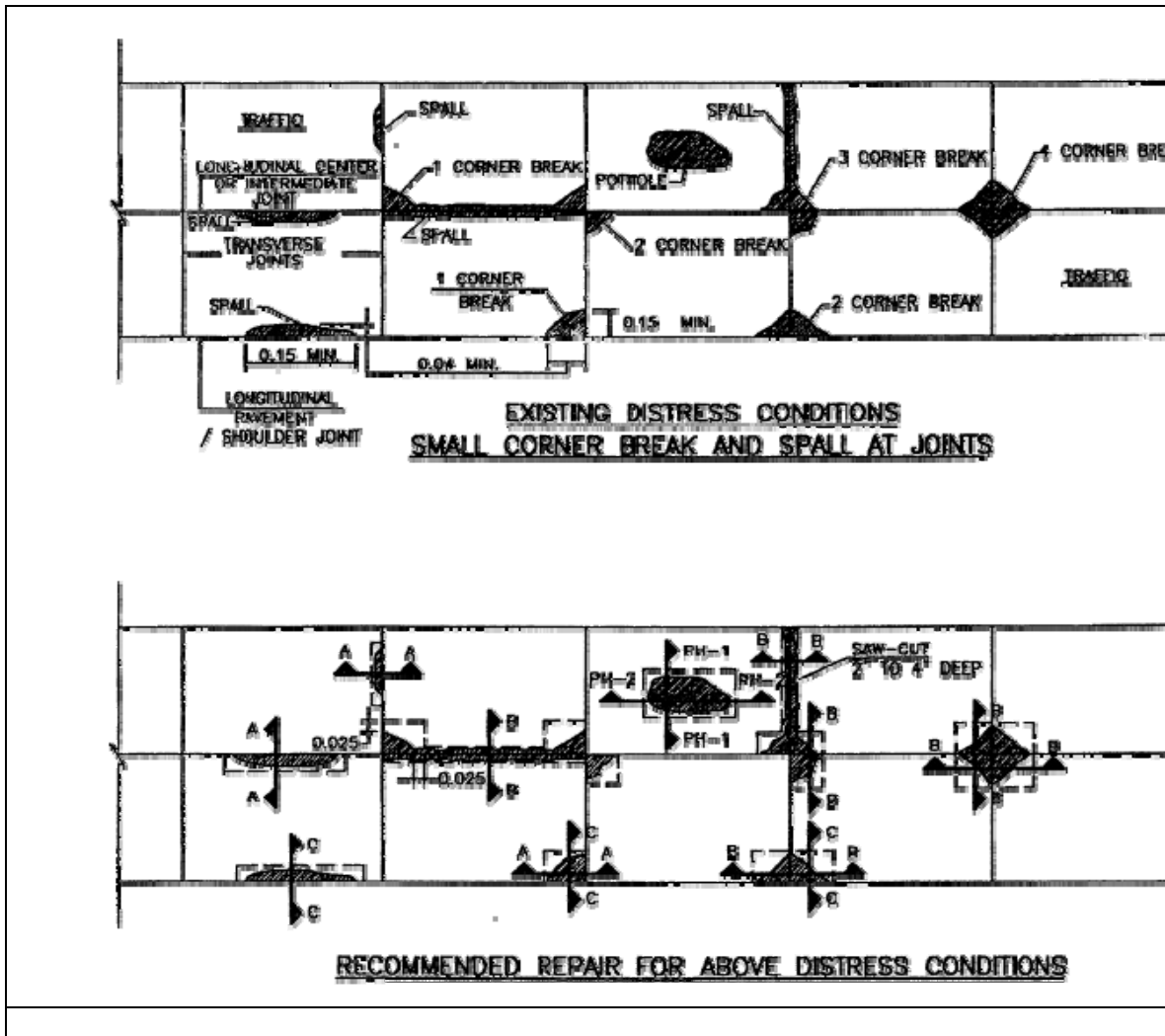


Figure 4 – Typical recommended repair

Due to the heavy traffic, special conditions were imposed on the contractor:

1. On Friday at 10:00 PM, the highway would be closed to traffic.
2. On Monday at 4:00 AM. The highway would be reopened to traffic
3. The performance specification required concrete with $f'_c = 3,000$ psi (20.6 MPa) at 5 days.

Due to the construction time restraints, demolition occurred through Friday night and preparation was finished during early morning Saturday. The concrete was poured beginning on Saturday morning up to 3:00 pm. Because the highway would be opened to traffic at 4:00 AM Monday rapid concrete strength development was needed within 40 hours after placement.

The approved mix design had very high cement content (846 lbs) in order to achieve the specified concrete compressive strength at 5 days. This solution yielded a concrete that will continue to increase strength for several weeks. Since the traffic would be opened before the recommended curing period, traffic would remove the curing compound, un-protecting the pavement.



Figure 5 – Plastic shrinkage cracks

After a few months of operation the pavement showed cracks on several of the repaired segments. Some of the cracks were superficial and some were full depth.

Several possibilities were considered to determine the probable cause of the cracking and possible alternatives to improve the new repairs of the pavement slabs. In addition to the construction procedures, the mix design and the repair design were evaluated.

Core samples were taken on several pavement slabs. All slabs were inspected to observe the type of distress shown. Combining the inspection with the study of the core samples, the slab segments were separated in groups indicating the type of damage. The evaluation was separated into:

1. Superficial cracks
2. Visible full depth cracks
3. Structural cracks due to cutting
4. Damaged joints
5. Excessive cutting



Figure 6 – Full depth cracks

Concrete Mix Design: Key to Excellence in Performance of New Concrete Structures

As a reaction to the cracks, the curing process was improved by combining the use of curing compound and burlap cure. This action improved the performance, but did not solve the cracking problem.

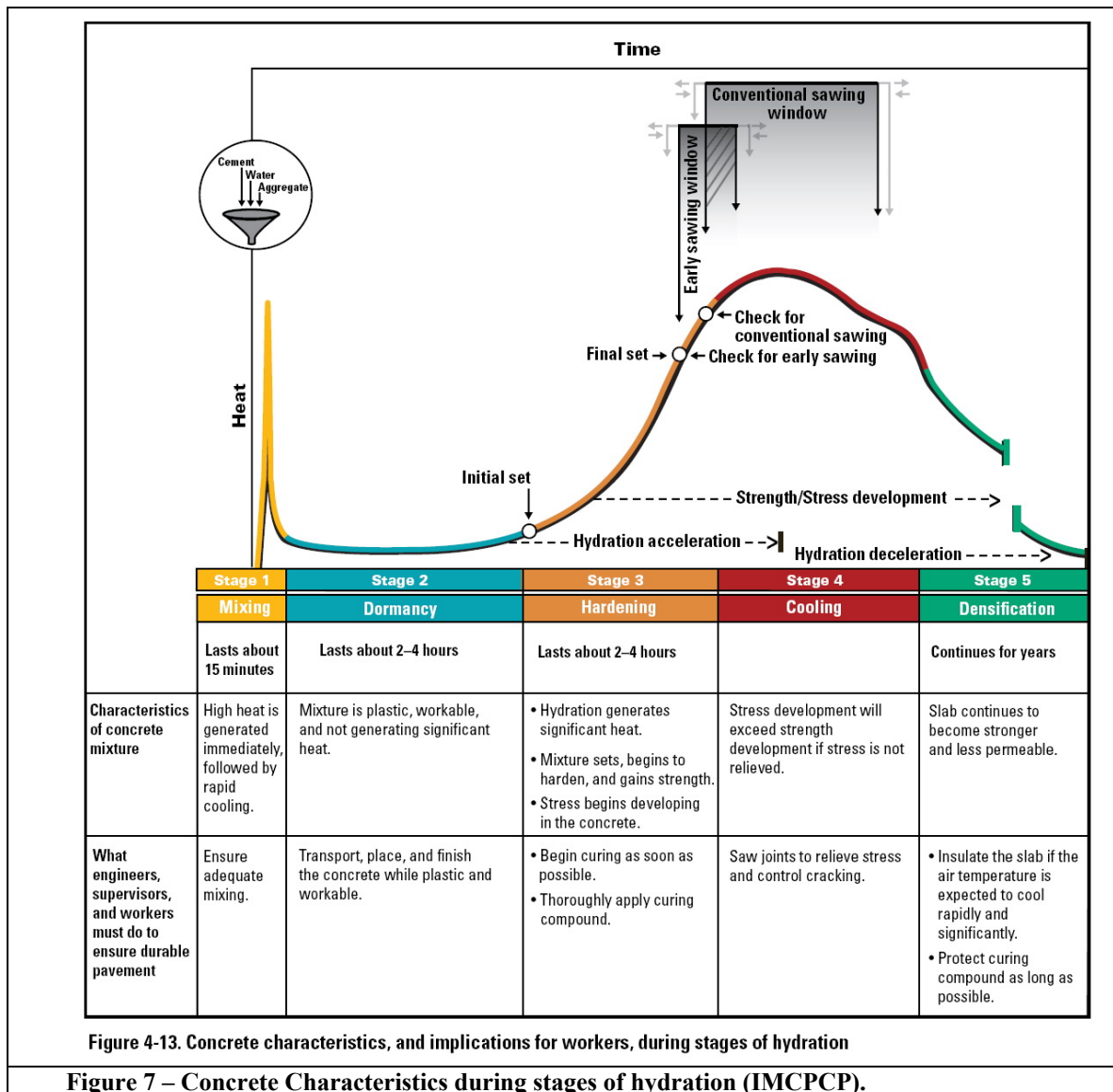
Other areas of potential problems were identified:

1. The project specified a reinforced concrete replacement slab, while the original one was unreinforced.
2. As shown in Figure 4, the plans show cutting segments within a slab to be cut out. This action would cause corner cracking of the remaining slab.
3. The new segment inside an existing slab would be connected by dowels on all sides, thus preventing temperature movement with the original slab.

The Federal Highway administration has developed a great publication for pavements, which includes pavement repair: Integrated Materials and Construction practices for concrete pavements (IMCPCP). It provides guidance in all areas of pavement design, construction and repair. In addition it covers cement and concrete for educational purposes. In addition it is very illustrative of the concrete hardening process (Figure 7).

In terms of the construction practice the IMCPCP manual gives three valuable recommendations:

- a. After sawing joints, insulate the pavement if air temperatures are expected to drop rapidly and significantly. (If the pavement surface cools too quickly, the slab may crack.)
- b. Keep traffic and construction equipment off the pavement as long as possible (preferably, at least 72 hours) to protect the curing compound. Curing compound reduces moisture loss from the concrete and thus enables continued hydration, increasing concrete strength and reducing *its* permeability.
- c. Tying a new concrete pavement to an existing pavement can result in significant stress on the new slab from restraint. This can be especially true if the new slab is placed on a warm sunny day following a cool night, as may occur in the fall. In the morning the existing slab is cool. As the day becomes sunny and warm, the existing slab warms and expands. If new concrete is placed and tied to the existing slab in the morning, it will reach the cooling stage of hydration and begin to shrink during the warmest part of day, when the existing slab's expansion is peaking. Because they are tied together, each slab's movement is restrained by the other's. The new, weaker pavement is likely to crack.



Several tests were performed for a new concrete mix design in order to develop a high early strength mix that would not continue to increase its strength significantly after five days. In addition tests were made considering the use of steel fibers instead of reinforcing bars. Based on the testing the final recommendations were:

1. Change the Concrete Mix Design to produce a concrete suitable at 36 hours without high volumes of cement.
2. Study alternative concrete mix designs with more compatibility with the existing pavement:
3. Eliminate reinforcement to match substrate properties.
4. Integrate partial slabs with repair materials and not dowels.
5. Change the shape of the cut sections for replacement, eliminating the creation of corners.
6. Schedule concrete placement to 4:00 am Saturday to force compatibility of expansion between existing concrete and new concrete.
7. Replace fully cracked slabs
8. Repair slabs with surface cracks with a High Molecular Weight Methacrylate (HMWM)

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SP-285 — 2

Replacement of 14 MG Reinforced Concrete Water Storage Reservoir and Pumping Station under a Golf Course: Planning, Design and Construction Challenges

By: Ashok K. Dhingra

Synopsis: This paper describes the planning, design and construction challenges to replace the existing, 80-year-old 14 Million Gallons (54.81 Mega liters) Chevy Chase Reservoir and Pumping Station, located in Glendale, California. The existing and new reservoirs were/are fully buried, reinforced concrete structures under a golf course in Chevy Chase Country Club. The new reservoir was built on the footprint of the existing reservoir, on a tight site, involving shoring and stockpile of earth to be utilized for backfill following the completion of the reservoir. The location is in a residential neighborhood along a busy street. There was heavy community involvement and the concrete truck traffic on pour days as well as the rebar deliveries had to be carefully managed during construction. This paper discusses the planning, pre-design, design, constructability review and the construction management including the various QA/QC aspects throughout the project. The inclusion of special provisions for demolition of the existing facilities and other special conditions are addressed. The existing reservoir did not last the anticipated life of 100 years for a buried structure, the lessons learned and how the new ACI 350 code was implemented into the new reservoir design will be discussed. Also, the leakage testing requirements are described.

Unforeseen subsurface conditions and unexpected contaminated perched groundwater along with the mitigation methods to improve soil and foundation conditions are discussed. The project was completed in record time, through an incentive clause in the contract documents. The project utilized the existing concrete material by crushing and using it as the sub base of the new reservoir, eliminating the need for hundreds of truck loads of material to be hauled offsite, contributing further to the Owner's "Green Initiative" and reducing the overall carbon foot print. Several unique features and the overall performance of the project will also be addressed.

Keywords: Construction, Demolition, Design, Green, Management, Planning, Reinforced Concrete Reservoir, Seismic, Tank,

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This paper was written and presented at the ACI 2010 Fall Conference, Pittsburgh, Pennsylvania for a tribute and symposium in honor of Dov Kamintzky

It was a privilege and honor to work with Dov on ACI Committees 350 and 364 and to have worked with him on a project that required his forensic expertise.

BACKGROUND

Originally constructed in 1928, this reinforced concrete reservoir with a wood roof had a capacity of 14.5 million gallons (54.81 Mega liters). The reservoir is currently known as the 968 Zone Chevy Chase Reservoir located in the Chevy Chase Country Club in an affluent community in Glendale, California. The reservoir is located east of Interstate 5 as well as State Route 2 and north of State Route 134 on Chevy Chase Drive as depicted in Figure 1 below.

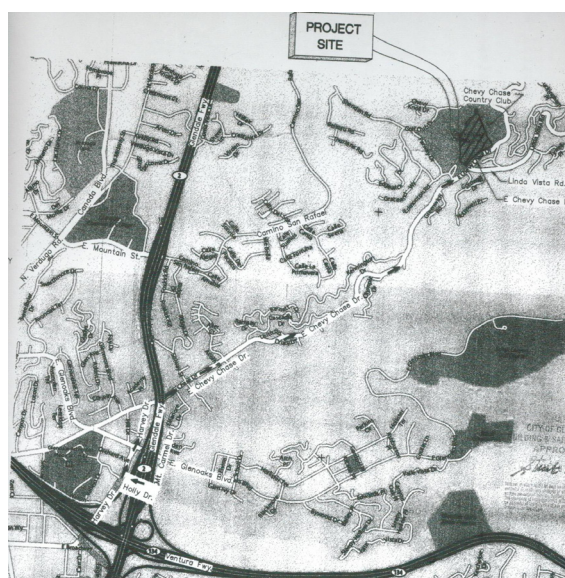


Figure 1 – Project site location

The 968 Zone refers to the elevation of the pressure zone of the water distribution system. Following a fire in the early 1930's the wood roof was replaced with a reinforced concrete roof. The reservoir had a hopper bottom with a liner on top and footings were cast on top of the liner and were interconnected with concrete struts formed and cast on top of the liner as shown in Figure 2 below. Subsequently, the reservoir experienced damage to the foundation system and columns during the January 1994 Northridge, California, earthquake due to the high vertical accelerations. Following this earthquake the codes were revised to require more stringent requirements to design for greater vertical accelerations.



Figure 2-- Interior of existing reservoir

DAMAGE TO EXISTING RESERVOIR

Following the Northridge earthquake, the City retained a Consulting Engineering firm to evaluate the damage caused by the earthquake. The interior of the dewatered reservoir was examined and in comparison to prior inspections the newly developed cracks due to the earthquake were discovered. The findings included damage to the isolated footings of the columns which had major cracks and the cracks in the struts interconnecting the footings. A photograph showing the typical cracks is shown in Figure 3.



Figure 3-- Cracks in the foundation system following the earthquake

The report also investigated the options to retrofit or rehabilitate the reservoir by pressure grouting the cracks in the columns and footings to provide additional life to the structure. Reconstruction of the roof was evaluated with internal shear walls and it was determined that both these options would not satisfy the seismic requirements of the current codes. A decision was thus made to replace the reservoir.

SITE ANALYSIS

Following the decision to replace the reservoir, the City retained another engineering firm to conduct an analysis of three separate sites. Considered were (1) utilizing the existing location, (2) a new location on the golf course and (3) another location in close proximity in the hills. The evaluation consisted of replacing the reservoir with another 14.5 million gallon (54.81 Mega liters) reinforced concrete reservoir at these sites or a combination of two tanks providing the total storage by constructing prestressed concrete wire wound tanks. There were thus a total of six alternatives. The City solicited public input during the environmental documentation preparation of the project. Ultimately the reservoir to be replaced at the existing location, with a similar reinforced concrete reservoir, was deemed to be the best choice.

PROJECT OBJECTIVES

Building the new reservoir at the same site as the existing reservoir required an upfront initial planning level effort to develop a plan to serve the residents with water while the existing reservoir was demolished and rebuilt at the same site. The following overall project objectives were thus developed prior to the predesign of the facility. Also listed are actions taken to achieve these objectives.

1. Meet water demands during summer with reservoir out of service only for the duration of one summer.
 - This was achieved by utilizing an incentive clause in the Construction Contract Documents to assure that reservoir is out of service only during one summer.
 - Operations plan included installing two Pressure Reducing Valve stations to supplement water from the zones above; this was done prior to construction.
2. Minimize traffic and noise impacts to the community. These were achieved by:
 - Requiring construction workers to park off -site and by utilizing illuminated signs to advise residents of important traffic and community meeting messages.
 - Incorporating a provision in the Contract Documents that no construction activity or equipment movement or other noise producing activity is allowed prior to 7 a.m.
3. Keep community informed of progress during siting, environmental reviews & construction.
 - This was achieved by holding meetings during the siting study, holding public meetings during the environmental documentation preparation, and by establishing a website and retaining a public relations consultant during construction.
4. Coordinate the work of design team, construction management team, contractor and City's operations and management team
 - This was done by the City retaining a Program Manager from a consulting engineering firm at the outset.

PLANNING CHALLENGES

Often, there is not enough time for thorough planning and there is a tendency to start design of the facility and construction immediately thereafter. In this case, there was a thorough evaluation of various retrofit options and site analysis conducted prior to getting to the preliminary design phase. Based on the objectives enumerated above and the initial work done so far, the following planning challenges were summarized for the preliminary design of the facility.

1. Reservoir is a critical facility to water system operations and thus could not be out for more than one summer, thus restricting the construction duration.
2. Retrofit options were not viable to comply with current codes, since the original construction was based on codes predating the 1933 Long Beach, California, which caused significant damage to buildings in Southern California and it was after that the codes began to incorporate design criteria to resist earthquake forces.
3. An earlier study conducted to evaluate three different sites with rectangular reinforced concrete reservoir and circular pre-stressed concrete wire-wound tank alternatives determined that the existing location with rectangular reinforced concrete reservoir was the superior alternative. This made it a difficult site which had a golf fairway on top of it, with the golf course obviously built after the construction of the buried reservoir, and the golf course had to be operational during the construction of the new replacing facility at the same footprint.
4. The existing pumping station constructed during the same era, housed in a brick building did not conform to the seismic codes and was to be a part of the replacement with the new reservoir, again having been constructed prior to the more stringent codes following the 1933 earthquake.

DESIGN CHALLENGES

1. Permitting: The existing reservoir was under the California Department of Water Resources, Division of Safety of Dams (DSOD) jurisdiction and would require decommissioning, and the new reservoir design may require permitting depending on the water height above the toe of slope. This would add to the construction time involving more coordinated inspections by DSOD for decommissioning and perhaps during construction of the new facility.
2. The City's requirement that the new buried reservoir must last 100 years and not 80 years as the existing one.
3. Incorporating the City's "Green Initiative" to recycle or reuse the construction material recovered from demolition, where practical.
4. Earthquake resistant design to avoid a repeat scenario of the 1994 Northridge Earthquake damage, incorporating the latest provisions relative to vertical acceleration during an earthquake.

Reservoir Design

The reservoir design consists of a 7-1/2 inches (190mm) thick floor slab placed on a 12- inch (300mm)- thick layer of drain rock with spread footings supporting the columns, cast integral with the floor slab. The reservoir floor has a small hopper at 5:1 horizontal to vertical slope with the flat portion of the floor slab being 25 feet (7.5m) away from the interior of the wall. The columns are 2 feet (610mm) round in diameter and supporting a

Dhingra

flat slab roof 12 inches (300mm) thick with drop panel thickness of 7-1/2 inches (190mm). The maximum earth cover on top of the roof is 2 feet (610mm). The overall dimensions of the reservoir are approximately 475 feet (145m) by 245 feet (75m) with columns spacing of 20 feet (6m) on center. The reservoir is divided into two basins with a common dividing wall. Seismic resistance is provided through shear walls which are 21 inches (535mm) thick. The design conforms to ACI 350 including seismic criteria supplemented with site specific geotechnical report. The leakage criteria are also per ACI 350.1, Tightness Testing of Environmental Engineering Concrete Structures.

Concrete

The compressive strength specified for concrete was 5,000 psi (35 MPa) for the entire reservoir work with a maximum aggregate size of 1-1/2 inches (38mm). The maximum water cement ratio specified was 0.4. The air content of concrete was specified as 3% at the point of placement with an allowable variation of plus/minus 1%. Maximum 28 day drying shrinkage was specified at .05%.

CONSTRUCTION CHALLENGES

One of the construction challenges was that the construction site was very tight since the footprint of the reservoir and pumping station was about 2.3 acres (.93 hectares), whereas the overall site was a little over 4 acres (1.62 hectares), as depicted in Figure 4. This was further compounded by the fact that the golf course had to be operational and there was space required to stockpile the excavated material for backfill as well as the staging and construction lay down area for Contractor's operations during construction.



Figure 4-- Excavated site, construction trailers in the foreground, L shape, along with stockpiled material for backfill covered with plastic

Replacement of 14 MG Reinforced Concrete Water Storage Reservoir and Pumping Station under a Golf Course

The tight site coupled with an aggressive schedule driven by the requirement of the reservoir being out of service for the duration of one summer required that the construction sequencing and work be done correctly with no margin of error to redo any work. It required close communication and coordination among the various parties throughout construction to accomplish this. The schedule was further exacerbated due to unforeseen conditions when contaminated (by hexavalent chromium) soils and water were discovered after the demolition of the existing reservoir. The situation was resolved by the City's Hazmat team and the contractor but resulted in a one month delay in the schedule. Another unforeseen item that impacted schedule was that the bedrock location turned out to be deeper than indicated by the geotechnical study, resulting in the placement of 30,000 cubic yards of soil stabilized material as opposed to 15,000 cubic yards anticipated. During demolition, noise issues had to be mitigated along with dust control as a result of crushing of the existing concrete, depicted in Figure 5.



Figure 5-- Demolition of existing reservoir in progress along with dust control mitigation

GREEN INITIATIVE

The City's requirement of "Green Initiative" which promotes sustainability by reuse of available materials and resources was achieved by reusing 8,000 cubic yards (5,867 cubic meters) of concrete from the existing structure and crushing it to be used as sub-base which is the recycled equivalent of 1500 truck loads, thus also reducing the overall carbon footprint. Furthermore, 6,250 tons (5,670 metric tons) of steel was hauled away for recycling, as can be seen in Figure 6.



Figure 6-- Steel stockpile for recycling and crushed concrete, half height dividing wall in the middle

DOCUMENTATION DURING CONSTRUCTION

The documentation of progress of work during construction is very important and cannot be overemphasized as the documentation is very helpful in resolving any issues or construction claims. The Contract Documents required the use of two cameras at the site which were monitoring the sites continuously and the live feed was available on the City's website. At the end of the project the City was able to compile a short video showing the progress of work. This project also required the contractor to take aerial photographs on a quarterly basis, as shown in Figure 7.



Figure 7-- Aerial photo February 11, 2009

RECONSTRUCTION OF GREENS AND FAIRWAYS

A major challenge also was planting grass for the fairways and greens of the golf course, at optimum time, to obtain desired growth at project completion. This time ought to be adequately accounted for in the overall construction schedule, since it is meticulous and tedious work. A reconstructed green is shown in Figure 8 with the construction manager evaluating the overall finish.



Figure 8-- Reconstructed green

PUMPING STATION

The existing pumping station was replaced with a new pumping station with two operating pumps at 1,150 gallons per minute (4353 liters per minute) at 432 feet (131.7m) of total dynamic head and a standby pump of the same capacity. The pumping station thus had 3- 200 HP pumps. The pumping station was located next to the reservoir such that one of the walls of the pumping station was a common wall of the reservoir. The walls of the pumping station up to the sub-grade level were reinforced concrete and above grade were of masonry block construction, for aesthetic reasons, as shown in Figure 9. A progress photo of placing the sod on two feet of soil cover over the reservoir roof is shown in Figure 10.

Replacement of 14 MG Reinforced Concrete Water Storage Reservoir and Pumping Station under a Golf Course



Figure 9-- Pumping station building and access to reservoir



Figure 10-- Aerial photo November 14, 2009, with sod placed on half the reservoir

WATERPROOFING

One of the essential elements in water reservoirs is to prevent any surface or rainwater from entering the reservoir containing potable water so as to avoid contamination. This is important as any of the rain water needs to be drained quickly. The roof of the reservoir with grass which is watered and fertilized periodically, as in the case of this golf course, needs to have a barrier so that any of the leaching of salts in the fertilizer does not get into the reservoir from the top. This was achieved by specifying membrane waterproofing on the exterior of the walls as can be seen in Figure 11. On top of the roof, a bituminous sheet membrane product, a self adhesive 60 mil (1.5mm) thick cold applied composite sheet consisting of 56 mils (1.43mm) of rubberized asphalt and 4 mils (.10mm) of cross laminated high density polyethylene film was utilized as shown in Figure 12.



Figure 11-- Backfill in progress against walls with membrane waterproofing



Figure 12-- Application of bituminous sheet membrane on roof prior to earth cover

LEAKAGE TESTING

The criteria specified for leakage testing was HST-075 per ACI 350.1-01, with no measurable loss of water. The structure was filled to a maximum operating level (overflow elevation) at the rate of 4 feet (1.2m) per day and maintained full for 48 hours prior to the beginning of the test to permit any initial absorption by concrete. Each of the two cells was tested individually. The test period was five (5) consecutive 24 hour periods. Daily measurements of air and water temperature and water level were taken. Each of the two cells passed the leakage test without any problems.

PROJECT QUANTITIES

Concrete and rebar

Approximately 13,500 cubic yards (9900 cubic meters) of concrete was placed in 230-24 inch (610mm) diameter columns, 308 slabs on concrete floor and 308 slabs for roof flat slab construction. Over 14,000 tons (12,700 metric tons) of rebar was used.

Earthwork

Approximately 30,000 cubic yards (22,000 cubic meters) of soil cement was used as base whereas 20,000 cubic yards (14,667 cubic meters) of soil was exported from site.

Dhingra

Reservoir volume

Reservoir volume of 14.5 million gallons (54.81 Mega liters) planned (nominal) - Actual calculated volume is 15million gallons (56.78 Mega liters), 2-7.5million gallons (28.39 Mega liters) basins.

Other

- Pumping Station: 2,300 gallons per minute (8,706 liters per minute) active, 1,150gpm (4353 liters per minute) standby, 3-200HP Pumps.
- 49 miles (78.4km) of waterproofing membrane 6 inches wide.
- 3 miles (4.8km) of piping.

LESSONS LEARNED

1. The decision made by the City to retain the Construction Management and Design teams at the same time and involving the Construction Management team in early design decisions and constructability reviews at various stages of entire project proved to be a good choice and this practice is encouraged, as a lesson learned.
2. For similar projects where a golf course or park is a part of the project, the involvement of a project manager representing that element throughout design and construction is essential for coordination and continuity, which was not the case in this project but was recognized at the conclusion of the project.
3. Operations staff involvement from the early stages is essential to describe the desired requirements to be incorporated in design and avoids any potential change orders, which proved very effective on this project.
4. In tight schedules, incentive clauses work. The Contract Documents definitions need to be crystal clear for all to have the same understanding, and these should be utilized for all major milestones, as was the case in this project.
5. Constructability reviews throughout the project are very valuable and especially on projects dealing with site constraints and aggressive schedules or operational constraints similar to this project.
6. A dedicated program manager to coordinate Owner's and Consultant's resources saves time and money as evidenced by the successful completion of this project.

CONCLUSION

The project was completed at a construction cost of \$23,341,000 including the incentive of \$500, 000 for substantial completion 60 days early. It was a very successful project because everyone from the contractor, design engineer, construction manager, program manager, and the owner's team communicated, coordinated and collaborated well throughout the construction duration.

ACKNOWLEDGEMENTS

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Tim Sarre, Project Manager, Sema Construction, Lake Forest, California

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

by Michael P. McGurl and David W. Johnston

Synopsis: Analysis of the construction load distribution during construction of multistory concrete buildings is critical for both construction safety and economy. Recent reduction of load factors used in design of the permanent structure has resulted in less strength availability in the construction stage and need for tools to better support shoring and reshoring analysis. Using the traditional simplified method presented in ACI SP-4 for concrete structures under construction, spreadsheet models have been developed which analyze the distribution of loads based on inputs chosen by the engineer. The models determine the vertical loads resisted by temporary construction systems and the slabs with various combinations of levels of shores and reshores and with mechanical drophead shores. Analysis can be performed using either unfactored or factored loads and using either actual uniform loads or loads expressed as a proportion of slab dead load. The results of the models were validated by comparison with previously published shoring and reshoring analyses. Lastly, a comparison of the loads supported by the individual slabs and the levels of shores in each model during the placement of upper floors is made.

Keywords: buildings, concrete, construction, formwork, reshoring, shoring

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INTRODUCTION

Due to the cost of material and labor, contractors design the construction process for multistory concrete structures to minimize the levels of shoring and reshoring and to minimize the time before stripping formwork. Therefore, the supported loads and load capacity of the slabs and shores must be known throughout all stages of construction to allow for the safe removal of shores and reshores. While methods have been developed to determine the vertical loads distributed to slabs and shoring systems during construction stages, the analysis process to determine loads given different combinations of shoring and reshoring has been time consuming or complex and consequently underutilized. Thus, contractors have continued to seek more efficient methods for the analysis and design of shoring and slab systems needed in multistory concrete construction. The objective of this study was to provide easy to use computer models, based on traditionally accepted methods, to facilitate the analysis process. Additionally, the occurrence of slab liftoff and its implications on load distribution which have not always been fully understood are considered.

By applying an analytical methodology developed in spreadsheet models for multiple combinations of shoring and reshoring systems, construction engineers can design safer and more cost effective temporary structures for multistory concrete building construction. Using the analysis systems developed in this study, construction engineers can quickly and easily input the live and dead loads applied to the structure to evaluate the loads carried by each slab and shoring level using either the traditional shores and reshores or drophead shores. The analysis can be performed using either unfactored or factored loads and using either actual uniform loads or loads expressed as a proportion of slab uniform dead load, D . By using the analysis results with appropriate system strength criteria, contractors will be better equipped to design a structurally safe and efficient construction cycle for future multistory concrete structures.

PREVIOUS SHORING ANALYSIS STUDIES

Previous research has provided proposed methods and recommendations for assumptions for the analysis of construction load distribution in multistory concrete structures under construction. Researchers have used experimental procedures and theoretical studies to attempt to quantify the construction loads that result in slab and shore systems. Multiple analysis techniques have been proposed to help construction engineers determine the patterns of load distribution that can be expected in structures under construction. The resulting analytical and computational models have improved the insight into the process for analysis and determination of efficient construction cycles.

In 1963, Grundy and Kabaila¹ developed some of the fundamental assumptions of what is often called the simplified method, an analytical procedure to determine the distribution of loads to slab and shoring systems. Modeling single bay structures to represent the construction of flat plate and flat slab concrete buildings, the authors showed, that while increasing the number of number of levels of shores would increase the peak slab load, the higher number of shored floors allowed concrete more curing time to strengthen in preparation for the additional loads. Furthermore, their analysis illustrated that, even if engineers considered the varying flexural stiffness of concrete slabs at different story levels, the resulting load distribution did not differ greatly compared with results based on the assumption of constant stiffness.

The results achieved by Grundy and Kabaila were considered in recommendations by M. K. Hurd and ACI Committee 347 beginning in the 4th edition of ACI SP-4 and continuing through the most recent editions^{2,3}. The committee presented the simplified analytical method after recognizing that a precise theoretical solution is

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

impractical for slab and shoring systems due to numerous factors created by creep of concrete, technique of installing reshores, local load redistribution, and degree of continuity in field-constructed formwork. Hurd states that, while the assumptions applied in the simplified method are not precisely true, the results of the simplified analysis can be justified by the small differences found in comparison with reported field measurements. Additionally, ACI 347.2R-05⁴ further presents a guide to the application of the simplified method. The basic assumptions listed in ACI SP-4 are:

1. All slabs are identical, and shores/reshores are aligned one-to-one from floor to floor.
2. Ground level or other base support is rigid and shores are spaced closely enough to treat the shore reactions as a distributed load.
3. Shores and reshores are infinitely stiff relative to the slabs.
4. Slabs are interconnected by shores, therefore all deflect equally when a new load is added, and carry a share of the load in proportion to their relative stiffnesses.
5. Slabs have equal stiffness. Thus, added loads are shared equally by interconnected slabs.

Several studies over the years have examined the assumptions and tried to determine the level of error introduced by the simplifying assumptions.

Having measured construction loads applied during a portion of the construction schedule of two flat slab buildings, Agarwal and Gardner⁵ determined the simplified method could predict the construction loads within 10% if variable floor stiffness was considered and within 15% with constant floor stiffness. Comparing load results with the SAP structural analysis program, Liu, Chen, and Bowman⁶ considered a reasonable modification coefficient for the simplified method to adjust for the errors introduced by its assumptions to be 1.05.

The results of the simplified method were further analyzed by Stivaros and Halvorsen⁷, who utilized the equivalent frame method (EFM) to measure slab loads in single-bay and multi-bay models. Using two levels of shores and one set of reshores, the researchers determined that, while differences in load results between the EFM and simplified model could be as high as 30%, the maximum slab load occurred during the same construction stage and was within 5% for the two methods. The leading cause for the differences in calculated loads was determined to be the assumption in the simplified method of infinitely stiff shores. The authors also noted that the results of all the methods, including those of Liu, Chen, and Bowman, converged following the removal of the last shored or reshored level from the foundation.

Comparing available methods with compiled data, Puente, Azkune, and Insausti⁸ concluded that the simplified method overestimates the shoring loads prior to the removal of the 1st level of shores by assuming infinite shore stiffness, which in turn under-predicts the loads carried by the slabs although this load is small. Following the removal of the shores between the 1st floor and ground, their data shows that the remaining shores, while carrying less loads, continue to transmit loads close to an individual concrete floors' dead load to levels below. At the identical construction step, the existing slabs in the simplified method support all construction loads from the shores.

As advancements were made in technology, engineers sought new ways to incorporate and improve established engineering methods with computers. Gross and Lew⁹ developed a microcomputer-based program coded in FORTRAN to assist in evaluating the safety and economy of alternate construction schemes for cast-in-place multistory concrete building construction. The purpose of the program was to assist contractors in the field to plan the most efficient casting schedule by varying the number of shored and reshored stories, the precompression of reshores, and the rate of construction. Similarly, Arafat¹⁰ developed 'RESHORE1' in FORTRAN to determine the optimal schedule for shore and reshore removal using daily inputs of current concrete temperature for construction stages. Recognizing that concrete columns and slabs in the same structure develop strength at a rate that is different from one floor to another, Arafat utilized the relationship between concrete strength gain and its age and temperature history in the analysis.

Liu, Chen, and Lee¹¹ developed a computational procedure to determine the construction loads supported by slab and shoring systems that, along with the corresponding program developed in FORTRAN, could be performed on a personal computer. In an attempt to advance the capability of the slab and shoring method, the authors included the effects of concrete maturity while allowing users to select the construction cycle times for the casting period, the curing period, and the total period from the placement of one floor until the next. However, users are limited to a range of three temperatures along with either Type I or Type II cement to represent the entire construction process. Offering more temperatures to represent the entire construction project while choosing between Type I or Type III cement concretes, Gardner and Muscati¹² developed 'RESHORE,' which relies more on the relationship between concrete shear strength and concrete cylinder strength while incorporating the effects of maturity.

While maintaining the assumptions of the simplified method, Lewis¹³ introduced the Drophead-beam-panel method to compare to the traditional simplified method. Despite incorporating different construction steps and load stages, results for the drophead method include the transfer of loads through the temporary structures with the same principles as the simplified method. Analysis for both methods was performed with developed Excel spreadsheet models which distributed construction loads in the slab and shoring systems. The contribution of the study reported here was first to refine and significantly expand those models.

CONSTRUCTION LOAD FACTORS

During the construction of multistory concrete structures, contractors must recognize the varying nature of the distributed loads and the factors that apply to them. When using ASD methods for design of formwork and shores, load combinations based on unfactored loads as presented in ACI SP-4 are appropriate. When using LRFD of formwork and shores or strength design of the slabs, engineers must determine the most appropriate factored load combinations depending on the stage of construction.

Written to provide minimum design load requirements for structures during construction, SEI/ASCE 37-02¹⁴ provides multiple categories of loads along with basic combinations. When determining the appropriate loading combination for the shoring and reshoring process, engineers and contractors often use:

$$U_C = 1.2D + 1.2C_D + 1.2C_{FML} + 1.4C_{VML} + 1.6C_P$$

where D = dead load of completed permanent construction;
 C_D = construction dead load (formwork, shores and reshores);
 C_{FML} = fixed material load;
 C_{VML} = variable material load; and
 C_P = construction personnel and equipment.

For design of formwork or supporting slabs during the temporary construction process, concrete is subject to two different load factors. According to SEI/ASCE 37-02, the weight of concrete placed in a form for the permanent structure is a material load. Localized loads, such as piled concrete during placing, should be considered to be a Variable Material Load, C_{VML} , particularly for design of specific formwork members on the floor being cast. Uniformly distributed concrete weight supported by formwork may be considered Fixed Material Load, C_{FML} . Consequently, for the shoring and reshoring analysis, the fixed material load factor of 1.2 appears appropriate but the higher factor of 1.4 for the variable material dead load could be used at the option of the construction engineer. However, once the concrete gains sufficient strength so that the formwork, shoring, and reshoring are not required for its support, the concrete becomes a permanent structure dead load, D . A load factor of 1.2 can be applied to the formwork, shoring, and reshoring because they are both fixed material dead load, C_{FML} , in some steps and temporary structure dead load, C_D , in other steps. The weight of personnel and equipment, C_P , is subject to a 1.6 load factor.

The results of the loading combination can be compared to the design strength established for permanent structures by ACI 318-08¹⁵ (and the 2005 edition), which requires $U = 1.2D + 1.6L$. This is a reduction from the requirement of $U = 1.4D + 1.7L$ for the design of permanent structures which existed previously. Thus, contractors responsible for the construction of buildings can no longer rely on the higher safety factors built into structures designed under earlier ACI 318 editions. By lowering the required strength of concrete in permanent structures, ACI 318-08 further increases the need for the contractor's accurate understanding of loads and their distribution during the stages of multistory construction.

ANALYSIS MODEL

While multiple systems of shoring and reshoring exist, the principle loads applied to them remain the same but vary in magnitude. In creating a model to determine the load distribution encountered during multi-level concrete construction, Table 1 was developed to maintain consistency throughout the stages of analysis and therefore the stages of typical construction. In the analysis, the individual loads are denoted with "i" representing the most recently constructed level within the stage. Using the labels provided, numerical equations used to monitor the vertical loads in theory and in the model can be more easily represented.

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

Table 1—Loads applied in shoring/reshoring analysis

Vertical Load	Simplified Method Symbol	Simplified Method Model Label
Fresh concrete fixed material load	Q_{CFML}	Fresh slab weight
Cured concrete slab dead load	Q_{CD}	Cured slab weight
Form/Shore dead load	Q_S	Form/shore weight
Reshore dead load	Q_R	Reshore weight
Construction live load	Q_L	Construction live load
Previous load resisted by slab	R_{CB}	Load carried by slab: beginning
New load resisted by slab	R_{CE}	Load carried by slab: end
Shore transfer load	R_S	Load carried by shore: end
Reshore transfer load	R_R	Load carried by reshore: end

While determining the means and methods of multistory concrete construction, construction engineers must consider the cyclical process of placing and recycling shores from level to level. The various slabs with varying accumulated concrete maturity both contribute to the loads in the shored system and support the loads. To safely and efficiently generate and monitor the shoring system, engineers must compare the strength of existing slabs with the vertical loads created by the construction process. Using the traditional simplified method for shoring and reshoring along with Microsoft Excel, AutoReshore models for the analysis of vertical loads experienced during the stages of construction have been developed. In addition, a model was created based on the drophead shore method. The occurrence and implications of slab liftoff are identified within the construction steps of the models to further focus the need to verify snugness of reshores during field operations or to use spring clips to prevent reshores from falling over.

Within the different AutoReshore models, which offer common combinations for levels of shoring and reshoring, input fields provide construction engineers the opportunity to analyze the structure using actual load magnitudes or loads in terms of slab weight, D . Load factors may be entered as 1.0 for allowable stress design or as appropriate for LRFD or strength design. Using the inputs, the models distribute the factored or unfactored loads on the structure to determine the necessary loads that must be supported by the slabs and shores during each stage of construction. The following assumptions are made in the analysis models:

1. The ground level support is assumed rigid. Therefore, slabs supported by shoring continuous to the ground do not deflect.
2. Shores are spaced closely enough to treat the shore reactions as a distributed load.
3. The shores and reshores are infinitely stiff. Therefore, slabs deflect equally when slabs bear on shores or reshores.
4. Slabs have equal stiffness. Therefore, as a result of 3 and 4, added loads are shared equally by interconnected slabs.

To differentiate between the AutoReshore models, each includes the combination of levels of shores and reshores within its title. For example, the model that includes the analysis for one level of shores and two levels of reshores is entitled AutoReshore 1S-2R. The following basic models were developed – 1S-2R, 1S-3R, 2S-1R, 2S-2R – plus A and B variations for differing orders of shore removal, and an additional model for the drophead shore system. In each case, the models extend to 10 floors by which the results are cycling at repeating values.

After selecting the model representing the combination of shoring and reshoring being investigated, the user must complete the inputs field at the top of the model (Figure 1). Each model contains input cells for the design loads and their load factors. The inputs determined by the user include “Fresh slab weight”, “Cured slab weight”, “Form/shore weight”, “Construction live load”, and “Reshore weight” all expressed per unit area of floor. Within

the construction steps, each elevated slab number includes the resulting loads supported by the slab at the beginning of each step. The change in supported load is then determined in the neighboring column followed by the load on the slab at the end of the step, which is the summation of the beginning load and the supported load change.

AutoReshore 2S-1R-A [Compatibility Mode] - Microsoft Excel															
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	
1	SHORING AND RESHORING ANALYSIS - ACI SP-4 METHOD - AutoReshore 2S-1R-A														
2															
3	INPUTS														
4															
5		Form sets	2			Reshore sets	1								
6															
7		Shore spacing				Reshore spacing									
8		N-S	4.000			N-S	4.000								
9		E-W	4.000			E-W	4.000								
10		Area	16.000			Area	16.000								
11															
12						L.F.	LOAD								
13		Fresh slab weight				1.000	1.000	1.000							
14		Cured slab weight				1.000	1.000	1.000							
15		Form/shore weight				1.000	0.100	0.100							
16		Construction live load				1.000	0.500	0.500							
17		Reshore weight				1.000	0.050	0.050							
18															
19															
20	Step #	Slab #	Load carried by slab			Load carried by shore						Individual shore load			
21			beginning	chg dur	end		end						end		
22	1 - A	1	0.000	0.000	0.000										
23															
24							1.600						26		
25															
26															
27	1 - B	1	0.000	0.000	0.000										
28															
29							1.100						18		
30															
31															

Figure 1—Inputs field and first construction stage in AutoReshore 2S-1R-A

The load supported by the individual floor's shores can be found in the row below the slab loads and is calculated from vertical equilibrium using the loads supported by the slabs at the end of the construction step. In addition, inputs can be provided for the spacing between shores and reshores allowing the model to calculate based on tributary area the individual shore and reshore loads. In the examples, a shore and reshore spacing of 4 ft. is used on both directions. The spacing can be different in the two directions and the reshore and shore spacings are often different in practice. Consistent units must be utilized. A short description and an image of the construction process are also provided onscreen beside individual steps.

To present an example of load distribution in the models, the input loads found in Table 5-3 of ACI SP-4 6th edition², which analyzes a shoring system with two levels of shoring and one level of reshoring, have been applied to the AutoReshore 2S-1R-A model. The applied loads in the system include: 1.000 D for slab weight, 0.100 D for form and shore weight, 0.050 D for reshore weight, and 0.500 D for construction live load. Comparing the analysis from AutoReshore 2S-1R-A with SP-4 6th edition, the results from the model prove to be similar and the numerical differences are only due to rounding to different numbers of decimals. The results from the AutoReshore model through the placement of the 5th floor slab can be seen in Figures 1-5.

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

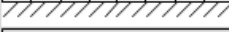
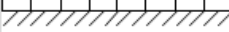
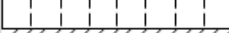
AutoReshore 2S-1R-A [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
32														
33	2 - A	2	0.000	0.000	0.000					2	Place			
34							1.600				2nd floor		26	
35											slab			
36														
37		1	0.000	0.000	0.000					1				
38														
39							2.700						43	
40														
41										2				
42	2 - B	2	0.000	0.000	0.000						Remove			
43							1.100				live load		18	
44														
45														
46		1	0.000	0.000	0.000					1				
47														
48							2.200						35	
49														
50										2				
51	2 - C	2	0.000	1.050	1.050						Remove			
52							0.050				1st floor		1	
53											shore			
54														
55		1	0.000	1.050	1.050					1				
56														
57							0.000						0	
58														
59										2				
60	2 - D	2	1.050	0.000	1.050						Place 1st			
61							0.050				floor		1	
62											reshores			
63														
64		1	1.050	0.000	1.050					1				
65														
66							0.050						1	
67														
68														

Figure 2—Second construction stage in AutoReshore 2S-1R-A

AutoReshore 2S-1R-A [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
69														
70	3 - A	3	0.000	0.000	0.000					3	Place			
71											3rd floor			
72							1.600				slab		26	
73														
74		2	1.050	0.000	1.050					2				
75														
76							1.650						26	
77														
78		1	1.050	0.000	1.050					1				
79														
80							1.650						26	
81														
82														
83	3 - B	3	0.000	0.000	0.000					3	Remove			
84											live load			
85							1.100						18	
86														
87		2	1.050	0.000	1.050					2				
88														
89							1.150						18	
90														
91		1	1.050	0.000	1.050					1				
92														
93							1.150						18	
94														
95														
96	3 - C	3	0.000	0.367	0.367					3	Remove			
97											1st floor			
98							0.733				reshores		12	
99														
100		2	1.050	0.367	1.417					2				
101														
102							0.417						7	
103														
104		1	1.050	0.367	1.417					1				
105														
106							0.000						0	
107														
108														
109	3 - D	3	0.367	0.158	0.525					3	Remove			
110											2nd floor			
111							0.575				shores		9	
112														
113		2	1.417	0.158	1.575					2				
114														
115							0.000						0	
116														
117		1	1.417	-0.417	1.000					1				
118														
119							0.000						0	
120														
121														
122	3 - E	3	0.525	0.000	0.525					3	Place			
123											2nd floor			
124							0.575				reshores		9	
125														
126		2	1.575	0.000	1.575					2				
127														
128							0.050						1	
129														
130		1	1.000	0.050	1.050					1				
131														
132							0.000						0	
133														
134														

Figure 3—Third construction stage in AutoReshore 2S-1R-A

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

AutoReshore 2S-1R-A [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
135														
136	4 - A	4	0.000	0.000	0.000					4	Place 4th floor slab			
137							1.600						26	
138														
139		3	0.525	0.533	1.058					3				
140							1.642						26	
141														
142		2	1.575	0.533	2.108					2				
143							0.583						9	
144														
145		1	1.050	0.533	1.583					1				
146							0.000						0	
147														
148														
149														
150														
151														
152	4 - B	4	0.000	0.000	0.000					4	Remove live load			
153							1.100						18	
154														
155		3	1.058	-0.167	0.892					3				
156							1.308						21	
157														
158		2	2.108	-0.167	1.942					2				
159							0.417						7	
160														
161		1	1.583	-0.167	1.417					1				
162							0.000						0	
163														
164														
165														
166														
167														
168														
169	4 - C	4	0.000	0.122	0.122					4	Remove 2nd floor reshores			
170							0.978						16	
171														
172		3	0.892	0.122	1.014					3				
173							1.064						17	
174														
175		2	1.942	0.122	2.064					2				
176							0.000						0	
177														
178		1	1.417	-0.417	1.000					1				
179							0.000						0	
180														
181														
182														
183														
184														
185														
186														

Figure 4—Fourth construction stage in AutoReshore 2S-1R-A

AutoReshore 2S-1R-A [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
186									////					
187	4 - D	4	0.122	0.482	0.604					4	Remove			
188							0.496				3rd floor		8	
189											shores			
190		3	1.014	0.482	1.496					3				
191													0	
192							0.000							
193														
194		2	2.064	-1.064	1.000					2			0	
195							0.000							
196													0	
197		1	1.000	0.000	1.000					1				
198							0.000							
199													0	
200														
201							0.000							
202													0	
203									////					
204	4 - E	4	0.604	0.000	0.604					4	Place			
205											3rd floor		8	
206							0.496				reshores			
207		3	1.496	0.000	1.496					3				
208														
209							0.050						1	
210														
211		2	1.000	0.050	1.050					2				
212													0	
213							0.000							
214		1	1.000	0.000	1.000					1				
215														
216							0.000						0	
217														
218													0	
219									////					
220														
221	5 - A	5	0.000	0.000	0.000				////	5	Place 5th			
222											floor		26	
223							1.600				slab			
224		4	0.604	0.533	1.138					4				
225														
226							1.563						25	
227														
228		3	1.496	0.533	2.029					3				
229														
230							0.583						9	
231														
232		2	1.050	0.533	1.583					2				
233														
234							0.000						0	
235														
236		1	1.000	0.000	1.000					1				
237														
238							0.000							
239													0	
240														
241							0.000							
242									////					

Figure 5—Fourth and fifth construction stage in AutoReshore 2S-1R-A

The application of actual design loads further illustrates the capabilities of the AutoReshore models as engineering design tools. While unit weight of concrete can vary depending on application, ACI SP-4^{2,3} states that the majority of all formwork involves concrete weighing 145-150 pcf (2320-2400 kg/m³). For design purposes, engineers often use 150 pcf (2400 kg/m³), which includes the weight of concrete and reinforcing steel. The weight of formwork varies from 3 or 4 psf (0.14 or 0.19 kN/m²) up to 10 to 15 psf (0.48 to 0.72 kN/m²). When considering the appropriate live loads, SEI/ASCE 37-02¹⁴ has established classes of working surfaces for combined uniformly distributed live loads. Medium duty considers concentrations of personnel along with staging of materials for average construction and requires the uniform live load to be 50 psf (2.4 kN/m²), consistent with the

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

recommendations of ACI 347-04¹⁶. For influence areas of 400 ft² or more, SEI/ASCE 37-02 provides for a live load reduction. To provide an example of AutoReshore analysis using actual design loads, Figures 6-10 illustrate a multistory concrete building designed with 8" (200 mm) slabs using one level of shoring and three levels of reshoring. The fresh concrete weight is considered a variable material load and the uniform weights of the forms/shores and reshores are estimated to be 10 psf (0.48 kN/m²) and 5 psf (0.24 kN/m²) respectively.

AutoReshore 1S-3R [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	SHORING AND RESHORING ANALYSIS - ACI SP-4 METHOD - AutoReshore 1S-3R													
2														
3	INPUTS													
4														
5		Form sets	1			Reshore sets	3							
6														
7		Shore spacing				Reshore spacing								
8		N-S	4.000			N-S	4.000							
9		E-W	4.000			E-W	4.000							
10		Area	16.000			Area	16.000							
11														
12						L.F.	LOAD							
13		Fresh slab weight	1.400	100.000	140.000									
14		Cured slab weight	1.200	100.000	120.000									
15		Form/shore weight	1.200	10.000	12.000									
16		Construction live load	1.600	50.000	80.000									
17		Reshore weight	1.200	5.000	6.000									
18														
19														
20	Step #	Slab #	Load carried by slab			Load carried by shore					Individual shore load			
21			beginning	chg dur	end		end					end		
22	1 - A	1	0.000	0.000	0.000									
23														
24							232.000						3712	
25														
26														
27	1 - B	1	0.000	0.000	0.000									
28														
29							152.000						2432	
30														
31														
32	1 - C	1	0.000	120.000	120.000									
33														
34							0.000						0	
35														
36														
37	1 - D	1	120.000	0.000	120.000									
38														
39							6.000						96	
40														
41														

Figure 6—Inputs field and first construction stage in AutoReshore 1S-3R

AutoReshore 1S-3R [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
42														
43	2 - A	2	0.000	0.000	0.000					2	Place			
44											2nd floor			
45							232.000				slab		3712	
46														
47		1	120.000	0.000	120.000					1				
48														
49							258.000						4128	
50														
51														
52	2 - B	2	0.000	0.000	0.000					2	Remove			
53											live load			
54							152.000						2432	
55														
56		1	120.000	0.000	120.000					1				
57														
58							158.000						2528	
59														
60														
61	2 - C	2	0.000	120.000	120.000					2	Remove			
62											shore			
63							0.000						0	
64														
65		1	120.000	0.000	120.000					1				
66														
67							6.000						96	
68														
69														
70	2 - D	2	120.000	0.000	120.000					2	Place			
71											2nd floor			
72							6.000				reshores		96	
73														
74		1	120.000	0.000	120.000					1				
75														
76							12.000						192	
77														
78														

Figure 7—Second construction stage in AutoReshore 1S-3R

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

AutoReshore 1S-3R [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
79														
80	3 - A	3	0.000	0.000	0.000					3	Place			
81											3rd floor			
82							232.000				slab		3712	
83		2	120.000	0.000	120.000					2				
84														
85							238.000						3808	
86		1	120.000	0.000	120.000					1				
87														
88							244.000						3904	
89														
90														
91														
92	3 - B	3	0.000	0.000	0.000					3	Remove			
93											live load			
94							152.000						2432	
95		2	120.000	0.000	120.000					2				
96														
97							158.000						2528	
98		1	120.000	0.000	120.000					1				
99														
100							164.000						2624	
101														
102														
103														
104	3 - C	3	0.000	120.000	120.000					3	Remove			
105											3rd floor			
106							0.000				shores		0	
107		2	120.000	0.000	120.000					2				
108														
109							6.000						96	
110		1	120.000	0.000	120.000					1				
111														
112							12.000						192	
113														
114														
115	3 - D	3	120.000	0.000	120.000					3	Place			
116							6.000				3rd floor		96	
117		2	120.000	0.000	120.000					2	reshores			
118							12.000						192	
119		1	120.000	0.000	120.000					1				
120														
121							18.000						288	
122														
123														
124														
125														
126														
127														
128														
129														
130														
131														

Figure 8—Third construction stage in AutoReshore 1S-3R

AutoReshore 1S-3R [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
132	4 - A	4	0.000	0.000	0.000					4	Place 4th floor slab			
133														
134							232.000						3712	
135														
136														
137	3		120.000	0.000	120.000					3				
138														
139							238.000						3808	
140														
141														
142	2		120.000	0.000	120.000					2				
143														
144							244.000						3904	
145														
146														
147	1		120.000	0.000	120.000					1				
148							250.000						4000	
149														
150														
151														
152	4 - B	4	0.000	0.000	0.000					4	Remove live load			
153							152.000						2432	
154														
155														
156							158.000						2528	
157	3		120.000	0.000	120.000					3				
158														
159														
160							164.000						2624	
161														
162	2		120.000	0.000	120.000					2				
163														
164							170.000						2720	
165														
166														
167	4 - C	4	0.000	120.000	120.000					4	Remove 4th floor shores			
168							0.000						0	
169														
170														
171														
172	3		120.000	0.000	120.000					3				
173							6.000						96	
174														
175														
176														
177	2		120.000	0.000	120.000					2				
178							12.000						192	
179														
180														
181														
182	1		120.000	0.000	120.000					1				
183							18.000						288	

Figure 9—Fourth construction stage in AutoReshore 1S-3R

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

AutoReshore 1S-3R [Compatibility Mode] - Microsoft Excel														
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
183									////					
184	4 - D	4	120.000	0.000	120.000					4	Remove			
185							0.000				1st floor		0	
186											reshores			
187		3	120.000	0.000	120.000					3				
188											Resnug		96	
189							6.000				reshores			
190														
191		2	120.000	6.000	126.000					2				
192							6.000						96	
193														
194		1	120.000	6.000	126.000					1				
195														
196							0.000						0	
197														
198									////					
199														
200	4 - E	4	120.000	0.000	120.000					4	Place 4th			
201							6.000				floor		96	
202											reshores			
203		3	120.000	2.000	122.000					3				
204							10.000						160	
205														
206		2	126.000	2.000	128.000					2				
207														
208							8.000						128	
209														
210		1	126.000	2.000	128.000					1				
211														
212							0.000						0	
213														
214									////					
215														
216														
217														
218	5 - A	5	0.000	0.000	0.000					5	Place 5th			
219											floor			
220							232.000				slab		3712	
221														
222		4	120.000	58.000	178.000					4				
223							180.000						2880	
224														
225		3	122.000	58.000	180.000					3				
226														
227							126.000						2016	
228														
229		2	128.000	58.000	186.000					2				
230							66.000						1056	
231														
232		1	128.000	58.000	186.000					1				
233														
234							0.000						0	
235														
236									////					
237														
238														
239														

Figure 10—Fourth and fifth construction stage in AutoReshore 1S-3R

While governed by the same assumptions described for the traditional simplified method of shoring and reshoring, the drophead shore method involves a different schedule of steps to complete each stage of the construction process. Following the erection of the formwork, concrete placement and initial curing of the concrete slab occurs. The beams and panels that support the new floor are then removed by lowering the dropheads without disturbing the shores. Consequently, the formwork can then be moved to the next level for erection while the first slab continues to gain strength prior to activation. Before the placement of the second level slab, the first slab, given the appropriate time to mature, is activated by retracting the shores, therefore transferring the loads to the concrete floor. The shores can then be converted to reshores by extending and snugging them under the activated level slab,

allowing for the distribution of future loads downward in subsequent construction steps. While the provided model utilizes one set of shores along with two levels of reshores, all of the shores are identical due to their conversion capacity.

Due to the different pattern of construction activities, the drophead shore model, AutoReshore 3DS, requires different inputs than the simplified model (Figure 11). For the different steps of slab construction, construction engineers must select appropriate loads for the “Fresh slab weight”, “Cured slab weight”, “Reinforcing/misc”, “Panel weight”, “Shore weight”, “Placing live load”, “Forming live load”, and “Reshore weight”. Similar to the previously described models, the drophead shore model allows the engineer to input specific loads or load ratios to be distributed throughout the stages of construction. Unlike the simplified method, the positioning of formwork and concrete reinforcing occurs during a separate construction step than the placement of any slab. As a result, the “Reinforcing/misc” load must be removed from the model when it is part of the weight of each newly placed concrete slab.

AutoReshore 3DS [Compatibility Mode] - Microsoft Excel															
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	SHORING AND RESHORING ANALYSIS - ACI SP-4 METHOD - AutoReshore 3DS														
2															
3	INPUTS			Panel Sets		1									
4															
5		Shore sets		1			Reshore sets		2						
6															
7		Shore spacing					Reshore spacing								
8		N-S	4.000				N-S	4.000							
9		E-W	4.000				E-W	4.000							
10		Area	16.000				Area	16.000							
11															
12						L.F.	LOAD								
13		Fresh slab weight				1.000	1.000	1.000							
14		Cured slab weight				1.000	1.000	1.000							
15		Reinforcing/misc				1.000	0.100	0.100							
16		Panel weight				1.000	0.050	0.050							
17		Shore weight				1.000	0.050	0.050							
18		Placing live load				1.000	0.500	0.500							
19		Forming live load				1.000	0.200	0.200							
20		Reshore weight				1.000	0.050	0.050							
21															
22															
23	Step #	Slab #	Load carried by slab			Load carried by shores							Individual shore load		
24			beginning	during	end		end							end	
25	1 - A	1	0.000	0.000	0.000										
26							0.400							6	
27															
28															
29	1 - B	1	0.000	0.000	0.000										
30							0.200							3	
31															
32															
33															
34	1 - C	1	0.000	0.000	0.000										
35															
36															
37							1.600							26	
38															
39															
40	1 - D	1	0.000	0.000	0.000										
41															
42							1.100							18	
43															
44															

Figure 11—Inputs field and first construction stage in AutoReshore 3DS

Spreadsheet Analysis of Load Distribution in Shoring and Reshoring Systems

While providing support to contractors and construction engineers during the design and construction of multistory concrete buildings, the AutoReshore models only determine the resulting loads on slabs and shoring throughout the construction process. Consequently, construction engineers must consider the loads imposed in the system in comparison to the strength available. As a concrete structure rises, the vertical loads created by newly placed concrete must be distributed through slabs that have yet to mature to their design strength. As a result, the distributed loads create bending in the slabs and shear at the supporting columns and shores.

OCCURRENCE OF SLAB LIFTOFF IN AUTORESHORE MODELS

When shores are removed from the system, the slabs below rebound upward. Under some situations, this can result in the slab or slabs lifting off the reshores. In practice, this is the reason that reshores sometimes can fall down and need to be resnugged if not braced and laced or secured in place with spring clips. While creep in curing concrete counteracts the developed separation between a slab and its reshores, creep is not considered in the simplified analysis. If the typical simplifying assumptions are applied without consideration of the lift-off issue, analysis results would incorrectly indicate some reshores in tension and slabs without applied load carrying more than their self weight.

In the AutoReshore analysis models, adjustments have been included through use of “conditional if” statements to identify and correct for lift off conditions. However, it is important that verification of reshore snugness occurs when these conditions exist in the shoring and reshoring process. To remind engineers of the preventative measure, additional reminder instructions have been included at the construction steps in the models where particular attention to resnugging of reshores may be needed.

AUTORESHORE VALIDATION

Following the development of AutoReshore, validation of the model was examined by comparing the outputs with previously published analysis results. ACI 347.2R-05⁴, ACI SP-4 6th edition², and ACI SP-4 7th edition³ present manually calculated examples of shoring and reshoring combinations with different input loads. In ACI 347.2R-05, the guide considers the construction cases in which one set of shores with either two or three levels of reshores are applied. The 6th edition of ACI SP-4 calculates load distribution in a shoring system with two levels of shores and one level of reshores and, unlike ACI 347.2R-05, includes reshore weight in the analysis.

While ACI SP-4 7th edition also considers the case for two levels of shoring and one of reshoring, it ignores reshore weight since it is usually small and presents a different order of construction steps following the removal of the construction live load from the 3rd floor. In the more recent SP-4 edition, the lower set of shores are removed prior to the set of reshores therefore relieving loads on the slab previously supporting the bottom set of shores and avoiding the additional load after the removal of the reshores. Consequently, AutoReshore 2S-1R-A and AutoReshore 2S-1R-B have been developed for engineers to analyze the construction sequences in the SP-4 6th and 7th editions respectfully. AutoReshore 2S-2R-A and AutoReshore 2S-2R-B apply similar shore and reshore removal patterns with the lower set of shores in the B model removed prior to the lower set of reshores.

By implementing identical conditions in the appropriate AutoReshore models, the outputs could be compared to the results from the ACI calculations. In both ACI and AutoReshore analysis, input loads were applied in terms of slab weight D . Due to the presentation of ACI results in terms of hundredths of D while the AutoReshore results are in thousandths of D , the outputs from the compared models vary only due to round off.

COMPARISON OF RESULTS FOR COMMONLY USED COMBINATIONS OF SHORES AND RESHORES

During the design of a construction schedule for multistory concrete buildings, engineers must consider the construction loads, concrete maturity, cost, and safety. Consequently, the rate at which subsequent levels of columns and slabs can be safely placed on an existing structure must be determined. The decision to construct each new level relies on the calculation of forces exerted on the existing slabs and shores so that the loads may be compared with the design strength of the shoring along with the developed strength of the in-place concrete.

To analyze the shoring systems at identical steps of construction in the models, construction loads resisted on individual slabs and their supporting shores were compared using load inputs in terms of slab weight D . Similar to

ACI SP-4², form/shore weight, construction live load, and reshore weight were selected as 0.100 D, 0.500 D, and 0.050 D respectively for the models with separate shores and reshores. To create the same construction load during slab placement, the drophead shore system incorporates reinforcing/misc. weight (0.100 D), panel weight (0.050 D), shore weight (0.050 D), placing live load (0.500 D), forming live load (0.200 D), and reshore weight (0.050 D). Consequently, the load supported by the 1st floor shores at the placement of slab 1 in each AutoReshore model is 1.600 D.

By analyzing the model results during the placement of slabs, the maximum loads supported in the shoring systems for these particular levels of applied load could be observed (Table 2). The lowest six floors from each model were considered because the loading pattern repeats following the construction stage that requires the removal of reshores from under the 1st floor. The largest slab-supported load is evident in slab 2 in both AutoReshore 2S-1R-A and AutoReshore 2S-1R-B during the placement of slab 4, which is the first floor placed in the 2S-1R models without construction load transferred to the ground. The maximum shore load is resisted by the 1st floor shores in the AutoReshore 2S-1R and 2S-2R models while the maximum reshore load is transferred by the 1st floor reshores during the placement of slab 4 in AutoReshore 1S-3R. In the 2S-1R and 2S-2R models, the lower set of shores must support the entire slab weight of both the 1st and 2nd floors along with the construction loads because slab 1 has yet to be released from its shores. During the placement of slab 4 in AutoReshore 1S-3R, the 1st floor reshores transfer the new construction load along with the weight of the structure's reshores to the ground.

Table 2—Comparison of model steps A that impose the largest loads on structural member i

Structural member i	Model step	Placed slab	AutoReshore model	Load result *
Slab 1	4 – A	Slab i+3 (4)	1S-2R	1.608 D
Slab 2	4 – A	Slab i+2 (4)	2S-1R A, 2S-1R B	2.108 D
Slab 3	5 – A	Slab i+2 (5)	2S-1R A, 2S-1R B	2.029 D
Slab 4	6 – A	Slab i+2 (6)	2S-1R A, 2S-1R B	2.069 D
Slab 5	7 – A	Slab i+2 (7)	2S-1R A, 2S-1R B	2.049 D
Slab 6	8 – A	Slab i+2 (8)	2S-1R A, 2S-1R B	2.059 D
Shores	2 – A	Slab i+1 (2)	2S-1R A, 2S-1R B, 2S-2R A, 2S-2R B	2.700 D
Reshores	4 – A	Slab i+3 (4)	1S-3R	1.750 D

* Based on live load = 0.50 D, Form and Shores = 0.10 D, Reshores = 0.05 D

CONCLUSIONS

The following observations and conclusions are based on the research conducted in this study.

1. Simplified models were developed in spreadsheets to analyze vertical load distribution in multistory concrete structures under construction using the traditional simplified method.
2. Sub-models A and B were developed for the shoring systems that include two levels of shores to simulate the two different shoring cycles recommended in ACI SP-4 6th and 7th editions^{2,3} for the removal of the lower set of shores.
3. Using identical load inputs, the model results from AutoReshore 1S-2R and 1S-3R are the same as the simplified analysis from ACI 347.2R-05⁴. Similarly, the results from AutoReshore 2S-1R-A and AutoReshore 2S-1R-B match the shoring analysis from ACI SP-4 6th and 7th editions^{2,3} respectively.
4. To assist in developing resultant load combinations carried and needed for design of the shoring system and the evaluation of the slabs, users may apply desired load factors to actual uniform loads or to loads in proportion to the slab dead weight.
5. The model's capacity to determine the direct vertical load distributed to individual shores and reshores further assists the design of shoring elements.
6. Improper load distribution resulting from slab liftoff can be properly mitigated by recognizing its potential within a shoring cycle. Consequently, notes have been included in AutoReshore models to verify snugness of reshores at appropriate construction steps prior to the placement of additional construction loads.

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Strengthening of Bridge by Pre-Stressing at JNPT and Testing its Efficacy

By Gopal. L. Rai, and A. N. Bambole

Synopsis: Jawaharlal Nehru Port Trust (JNPT) manages one of the busiest port in india, Nhava- Sheva port. This port handles 65% of India's container traffic has a berthing period of 37 hours which is considered to be very long. GTI one of the three container terminals in JNPT has set national record for berth productivity of 236 moves/hour on January 25, 2010 with the vessel M.V EVER RACER (WCIX service). The entire operation was performed in just 13 hour and 10 minutes, during which 3,295 moves (4,136 TEUs)¹ were performed. The Rail over bridge (ROB) has heavy vehicles moving day and night all the time. The bridge is crucial for functioning of the port if it is stopped it will make the container transportation very long and time consuming. The bridge had started showing cracks and distress.

This paper describes the detailed repairs that were carried on this bridge. An elaborate instrumentation system was installed to check the effectiveness of the repair work and both the static and dynamic load tests were carried out. The paper presents results obtained by the tests and sensors installed at the bridge before and after the repair work carried out for future monitoring of the structural health (SHM), which is believed to be an important feature of such work in India probably for the first time in India.

Keywords: Rehabilitation, Bridge, SHM, Structural health monitoring, JNPT

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ACI India Chapter Member Dr. A. N. Bambole is Professor at VJTI, Mumbai, India. He completed his Post Graduation from SPCE and Ph.D. from IIT Bombay, Powai. His area of interest includes Composite Mechanics, Finite Element Method, Non-Destructive evaluation techniques, Fiber Reinforced Composites in Repair, Retrofitting and Strengthening of structures, Fire Endurance of RC structures, and Waterproofing of RC structures. He recently got selected as a "Senate Member" in Mumbai University. He is a member on board of management of college as per Statute for autonomous institutes. He has played wide role in development of laboratories and other facilities in the college. Dr. A. N. Bambole has also been involved in many consultancy works some of the clients are JNPT, National Highway Authority of India, Indian Oil Corporation, MMRDA, BSNL etc. He has also been instrumental in organizing conferences and seminar in IIT Bombay and other Colleges in Mumbai. He is presently an active member in Indian Society for Technical Education (ISTE), Indian Society for Steel Development and Growth (INSDAG), American Society of Civil Engineers (ASCE), Indian Society for Non-Destructive Testing (ISNT) and Association for Structural Rehabilitation (ASTR) where he holds the position of secretary Maharashtra chapter.

INTRODUCTION

In today's growing economy, Infrastructure development is also raising its pace. Many reinforced concrete and masonry buildings are constructed annually around the globe. A is a large number of them, are deteriorated or become unsafe to use because of changes in use, changes in loading, change in design configuration, inferior building material used or natural calamities. Thus repairing and retrofitting these structures for safe usage of these structures has a great market.

Port at Nhava-Sheva, Navi Mumbai is managed by the Jawaharlal Nehru Port Trust (JNPT) [1]. This port is one of the busiest port and handles about 70% of container traffic of whole India has constructed Rail over bridge at Karal (Fig. 1) for efficient traffic flow. The construction of this bridge was completed and opened for traffic since 1991. This bridge consists of 36 spans of varying lengths with 37 expansion joints. The total length of bridge is 2297 m (7536.1 ft).

Depending on the desired properties, usage and level of damage involved members can be repaired and/or strengthened by several widely used methods.

One of the methods to strengthening is concrete jacketing but it has a lot of limitations [2]. The jacket should be of minimum thickness 100mm (3.94 inch). The sizes of members are increased and the free available usable space

Strengthening of Bridge by Pre-Stressing at JNPT and Testing its Efficacy

becomes less also adding a huge dead load and increasing the stiffness which reduces the efficiency of the structure. Its durability has also often found to be limited. Furthermore the whole process is slow and takes lot of time for completion.

Jackets may also be made of steel [2]. It's a popular technique to use steel plates bonded with epoxy to external surfaces of beams and slabs. This technique is simple and effective as far as both cost and mechanical performance is concerned, but suffers major disadvantages. Corrosion of steel plates hurdles its use in structures in/near river, lake and sea. Furthermore difficulty in manipulating heavy steel plates in tight construction sites, need for scaffolding, and limitations in available plate lengths which results in need of joints. Sometimes higher modulus of steel causes it to take large portion of axial load resulting in premature buckling.

The conventional jackets, sheets, plates may be replaced with Fiber Reinforced Polymer (FRP) fabrics, sheet and laminates in view of above limitations [3].

The pre-stressing of concrete is known to be very effective way of using the higher compressive strength property to much greater extent. Moreover, permanent deformations in the structure can be recovered by this technique. This technique of prestressing concrete is possible only in new structures. External prestressing with other materials of the existing structures have always been difficult especially in view of the materials to be used, reinforcement corrosion, lateral instability, end anchorages and of course space constraints.

The advantages of resistance to corrosion and high specific strength make these materials ideal for reinforcing existing structures with minimum intrusion. Popular method adopted is bonding them adhesively to concrete structures [4]. However, we can seldom fully use the superior strength properties of these Fiber Reinforced Composites (FRC's) due to poor capacities of concrete and interfaces formed. Pre-stressing of these materials allows us to better utilize these properties.

DETAILS OF THE BRIDGE

The Rail Over Bridge at Karal had heavy traffic movement and had started showing signs of distress. Layout of the bridge is shown in Fig. 2, the portion marked by blue line is a slab-girder (T-Beam) bridge system and the pre-stressed slab system portion marked in red. The bridge was functionally designed for crossing railway line and was structurally designed for Indian Road Congress (IRC) 45R loading. As per the revised recommendations of IRC and prevailing practice, the bridge is now required to sustain IRC Class 70 R loading.



Figure 1. ROB (Rail Over Bridge) at Karal Junction, JNPT

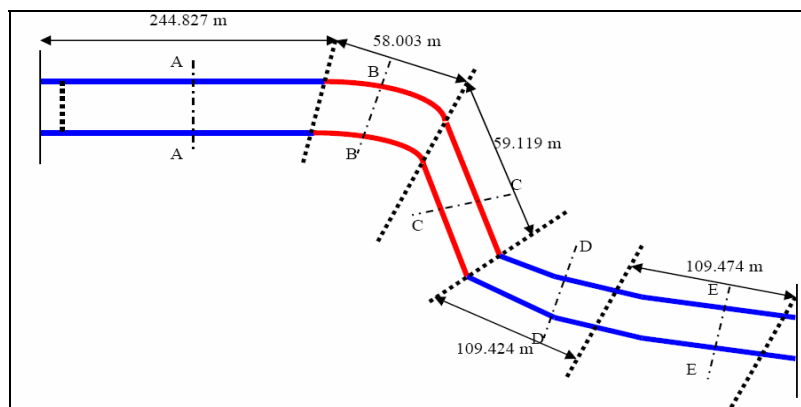


Figure 2. Alignment of the Karal Bridge

INSPECTION FINDINGS

To assess the information about the bridge current status and the repair methodology as it was inspected by consultants and structural experts.

The major observations during the site visit were as follows.

1. Expansion joints were not functioning properly. In the original design there was no provision of appropriate expansion joint.
2. Slab area of expansion joint was found to be damaged severely due to heavy vehicular movement. The gap between two spans has become significant and concrete had deteriorated.
3. The neoprene/elastomeric bearings provided in bridge were inadequate for heavy vehicle movements. They appeared to be bulging out and damaged.
4. There was a visible sag in the superstructure in many spans. The typical structural failure cracks in the girders were observed.
5. The new expansion joints could not last long due to excessive vibrations and the poor quality of deck concrete at the end of span.
6. The substructure/piers appeared to be sound.

PROCEDURE FOR REHABILITATION [5]

In view of above observations it seemed that the structural health of the bridge was not very good. The proposed strengthening measures for the bridge by the consulting team are as follows.

1. Strengthening of girder by steel truss system-

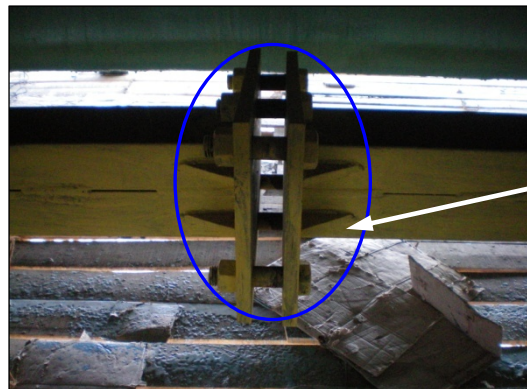
The girders and slabs are to be strengthened by placing additional steel truss systems, which will support the bridge deck/slab/girders with M32 high strength bolts. This was designed to take about 50% of load carrying capacity of girders. (Figure 3)

2. Replacement of bearings

The existing neoprene/elastomeric bearings should be replaced by new elastomeric bearings. Shore-A-hardness hardness of rubber material used should be 60. (Figure 4)



(a) Connection of Truss over Diaphragm Beam



**Pre-Stress of 100 Mpa
(14500 psi)**

(b) Pre-stressing Mechanism in Truss

Figure 3(a) and 3(b) Installation of Steel Truss System



Figure 4 - Replacement of damaged Bearings with new

3. Provision of new expansion joints-
It was recommended to replace the expansion joints with WaboCrete Strip Seal Expansion Joint System. It is a superior joint system, which can be rapidly installed in failed expansion joints and also is suitable for heavy vehicle bridges.
4. Pre-Stressing using C-fiber Laminate and Carbon fiber wrapping of girder and slab:
To further increase the structural strength of the bridge, it was recommended to strengthen the bridge using the Carbon fiber composite wrapping around the girder and slab. At the bottom of each girder 3 Pre-stressed Carbon Fiber Composites (CFC) laminates i.e. 2- 80/1.4mm (3.14/0.055 inch) and 1- 50/1.4mm (1.97/0.055 inch) was proposed to be placed. The load to be given to prestressed laminates should be 8-9 tons (17637-19842 Pounds). The deck slab was also recommended to be strengthened by putting CFC laminates 80/1.4 mm (3.14/0.055 inch) at 50 mm (1.97 inch) c/c at the bottom. The properties of required laminates and wrap were specified. (Figures 5 and 6)



Figure 5. Pre-stressing in process and the finished laminate with anchor plates at end.