DEFLECTION RECOVERY OF FIRE DAMAGED PRECAST REINFORCED CONCRETE RIBBED ROOF SLAB

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ABSTRACT

This paper discusses deflection recovery of a precast reinforced concrete (RC) ribbed roof slab after the application of a 24 hour loading test in the determination of integrity of the slab that was affected by fire in Dar es Salaam Tanzania. The loading was achieved by using sand filled sacks. The average load recovery was within the BS 8110 acceptable range of above the limit of 75%, with the exception of the middle strip zone which did not attain the limit. From the results, it was observed that the RC precast ribbed slab has to be divided into structural zones in relation to its structural configuration. Therefore, interventions need to be applied by considering each structural zone separately when assigning corrective measures, which will be specific on extent of strengthening for each structural zone instead of general corrective measures to the whole precast RC ribbed slab. Based on the mean deflection recovery test results, the slab was found to be still strong and acceptable for service.

Key Words: load testing, precast RC ribbed slab, deflection recovery

1.0 INTRODUCTION

The need to conduct structural integrity tests on reinforced concrete buildings arises when it is required to save human life and buildings, to rehabilitate them or to increase their life span after deterioration or damage due to fire accident, fatigue loads, chemical attacks and earthquake. Structural integrity of a building can be determined through destructive or non-destructive testing procedures (Mahadikand et al, 2014).

Several techniques have been used on evaluation of structural integrity of structures using different non destructive methods some of which use the state of the art technologies such as acoustic emission techniques, radioactive and nuclear methods, sonic and ultrasonic methods, penetration tests, load tests, rebound hammer test and artificial intelligence (Saleem et al, 2012, Casadei et al, 2005, Mahadikand et al, 2014, Lucas et al, 2014, Samarin et al, 1981, and Hola et al, 2005).

Load test being one of preferable technique is used to determine the integrity of a structure. For example in Pakistan, Saleem et al. (2012) conducted a non-destructive load test and partially destructive core test on an existing five storey concrete framed structure whose construction was halted for seven years and concluded that load test is more conclusive than core test and should be preferred. In another study, a 24 hour load test based on ACI 318-02 was compared with a cyclic load test by Casadei et al (2005). Both tests were applied on two identical slabs and produced the same outcome proving the validity of in situ load testing for determination of structural integrity of building members. The ACI 318-02 and other studies

recommend for more than one test be performed for evaluation of the integrity of a building (Samarin et al, 1981, and Hola et al, 2005).

This paper presents the results of load test deflection recovery of a precast reinforced concrete (RC) ribbed roof slab after the application of a 24 hour loading in the determination of integrity of the slab that was affected by fire in Dar es Salaam Tanzania. The acceptance criteria used was deflection recovery (residue elasticity) as per BS 8110-2 and the limit is 75% recovery. In this study observations were made on behaviour of different locations of the slab with different structural configurations.

2.0 THE STRUCTURAL SYSTEM AND PHYSICAL CONDITION OF THE STRUCTURE

The roof was composed of precast reinforced concrete slabs supported on in-situ cast reinforced concrete columns through the stiffened and deepened edges of the slabs. The edges of the slabs form a system of ribs at a spacing of 1800 mm c/c as shown in **Error! Reference source not found.**. The edge ribs of the slabs were 230 mm deeper than slabs. The exterior columns are of dimension 200 mm x 400 mm and the interior columns are of dimension 200 mm x 600 mm. The span of the slab panels between beams is 4.6 m. The precast slab panel is of dimension 4.73 m x 1.8 m and its other dimensions are as shown in Figures 1 and 2.

The slab was exposed to fire for about 3 hours before the fire could be extinguished. The soffit of the building was observed to exhibit spalling of concrete cover with exposed reinforcements at different locations as seen in Figure 4.

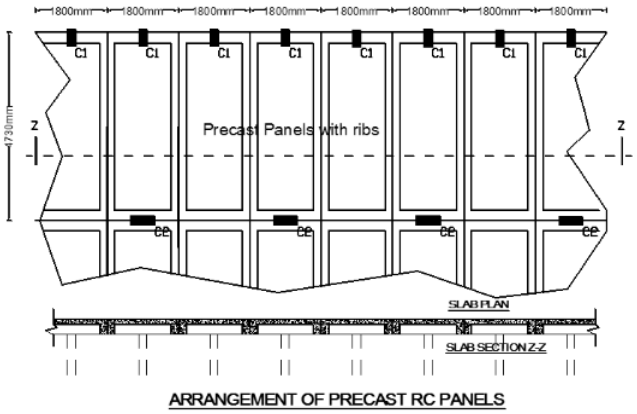


Figure 1: Precast RC slab panels arrangement

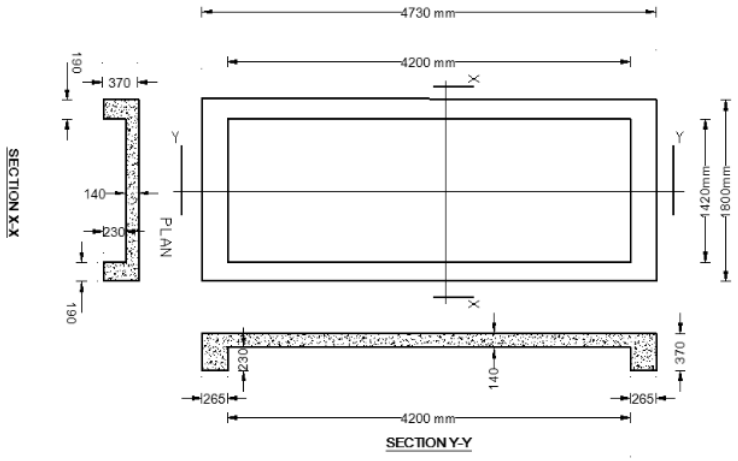


Figure 2: Dimensions of the precast RC panel

3.0 METHODOLOGY

3.1 Discrete loads using filled sand jute bags

In this case in-situ load test was conducted on a roof slab located above the 14th floor of a building where it was most deteriorated. The load test was done according to specifications and procedure given in BS 8110; 1985; Parts 1 and 2. The required Test Load N_{ult} for subjecting the structure into the test is one and a quarter times the superimposed load used during the design as expressed by equation 1. In this case, for an office building, the superimposed load Q_k was 5 kN/m^2 which then yielded 6.25 kN/m^2.

$$N_{ult} = 1.25\ Q_k \quad (1)$$

Where,
Q_k= Design Superimposed Load in kN/m^2
N_{ult} = Test Load in kN/m^2

Usually a structural steel loading frame system with hydraulic mechanism is used to conduct the loading test. Due to the weight and size of the required loading frame and obstructions, there are challenges to carry it to the target floor without causing damages on the way up. The alternative which is also very cumbersome was to disassemble the frame before carrying it up, then reassemble it on the concerned slab, disassemble it again to take it down and assemble it back.

Sand load was chosen for the purpose due to its ability to provide a uniformly distributed load. A total load of 53.2 kN (5.32 tons) of sand was placed on one slab panel with a size of 1.8 m x 4.73 m (Area, A =8.514 m^2) to provide a distributed load of 6.25 kN/m^2 as per equation 2. To ease the loading and off-loading processes, the sand was filled in 166 jute bags each filled with 32 kg of sand and one bag filled up to a quarter then closed up using sisal ropes as shown in Figure 3.

$$P = N_{ult} \cdot A \quad (2)$$

Where,
P = Total Test Load in kN

N_{ult} = Test Load in kN/m^2
A = Loaded Surface Area of the slab in m^2

A total number of 25 casual labourers were required for 4 hours to fill the bags with sand, to carry the bags from the ground floor, transport them in elevators, offload them from the elevator and hoist them to the roof slab over the 14th Floor. To avoid impact load effect to the slab, the bags were laid down gently. Close monitoring was required during filling of the bags with sand and while laying them down.



(a) (b)

Figure 3: (a) Sand loading process on the roof slab, (b) Sand loading in position



Figure 4: View of the slab soffit

3.2 Spot Levels

Before loading, spot levels were taken at predetermined points on the edge line, the centreline and in between as shown in Figure 5. Then the prescribed loading of 6.25 kN/m^2 was applied on the structure, after finishing the loading, the structure was left under load for a duration of 24 hours. After the 24 hours, levels at the respective points were taken for the purpose of determining the deflection between no-load and under-load conditions and then the structure

was off-loaded and left to relieve for another 24 hours, after which the final levels were taken in order to determine the amount of recovery from deflected condition due to the prescribed load. The levels were taken at the same respective points. The off-loading of the sand bags was done for a duration of about 3 hours. The loading sequence and slab response is as simulated in Figure 6.

The levelling instrument used was an automatic level Ni3 with a parallel-plate micrometer having an accuracy of ±0.0001 m. The reading error can be estimated at ±0.0002 m. A total number of 45 points were located and the levels taken. The establishment of the level was referenced from a temporary bench mark that was a fixed location on another slab panel which was not affected by the loading test.

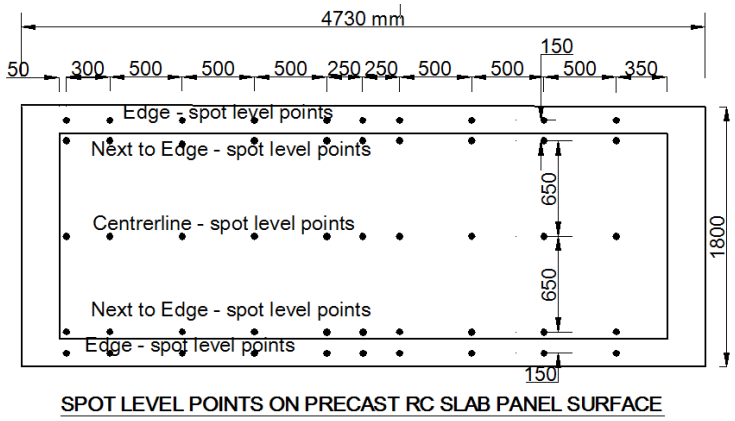


Figure 5: Precast RC slab panel showing spot level points

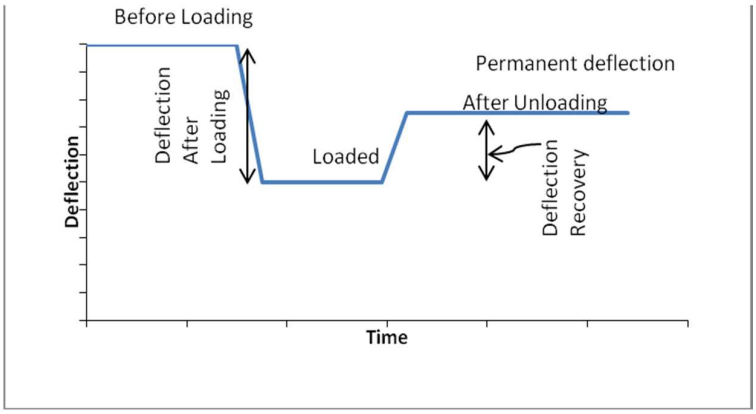


Figure 6: Loading Sequence and precast slab panel response

4.0 RESULTS AND DISCUSSIONS

4.1 Deflection due to application of the load

The results of deflection after 24 hours of loading are summarised in Table 1 and Table 2 for the slab panel centreline, slab panel edges and next to precast slab panel edges due to the configuration of the slab.

Precast slab panel centre line: The deflection of the precast slab panel centreline after 24 hours of loading reached a maximum of 53.2 mm, a minimum of 32.7 mm and an average deflection of 47.2 mm.
Precast slab panel edges: The deflection of the precast slab panel edges after 24 hours of loading reached a maximum of 53.2 mm, a minimum of 32.1 mm and an average deflection of 47.6 mm.
Next to precast slab panel edges: The deflection of the precast slab panel next to edges after 24 hours of loading reached a maximum of 56.0 mm, a minimum of 32.3 mm and an average deflection of 47.1 mm.
Total deflections: The total deflections of the precast slab panel after 24 hours of loading reached a maximum of 56 mm, a minimum of 32.1 mm and an average of deflection of 47.3 mm.

Table 1: Average precast slab panel deflection after 24 hours loading

Position	Deflection in mm			Condition	Remarks
	Mean	Minimum	Maximum		
Along edge of precast slab panel	47.6	32.1	53.2	>18.9 mm	Exceeds serviceability limit
Along next to edge of precast slab panel	47.1	32.3	56.0	> 18.9 mm	Exceeds serviceability limit
Along centreline	47.2	32.7	53.2	> 18.9 mm	Exceeds serviceability limit
Whole precast slab panel	47.3	32.1	56.0	> 18.9 mm	Exceeds serviceability limit

Table 2: Maximum and Minimum precast slab panel deflection

Position	Deflection	
	Minimum	Maximum
Along edge of precast slab panel	32.1	53.2
Along next to edge of precast slab panel	32.3	56.0
Along centreline of precast slab panel	32.7	53.2
Whole precast slab panel	32.1	56.0

The deflection graphic results are shown in Figure 7 and the deflections are consistently all above the highest limiting deflection value of 26.3 mm (L/180) for a roof slab according to ACI 318-14 and 18.9 mm (L/250) according to Eurocode 2 and BS 8110. Hence there was excessive deflection due to the design loading specified in BS 8110-1 and BS 8110-2. This an indication that the slab has been weakened due to the fire attack.

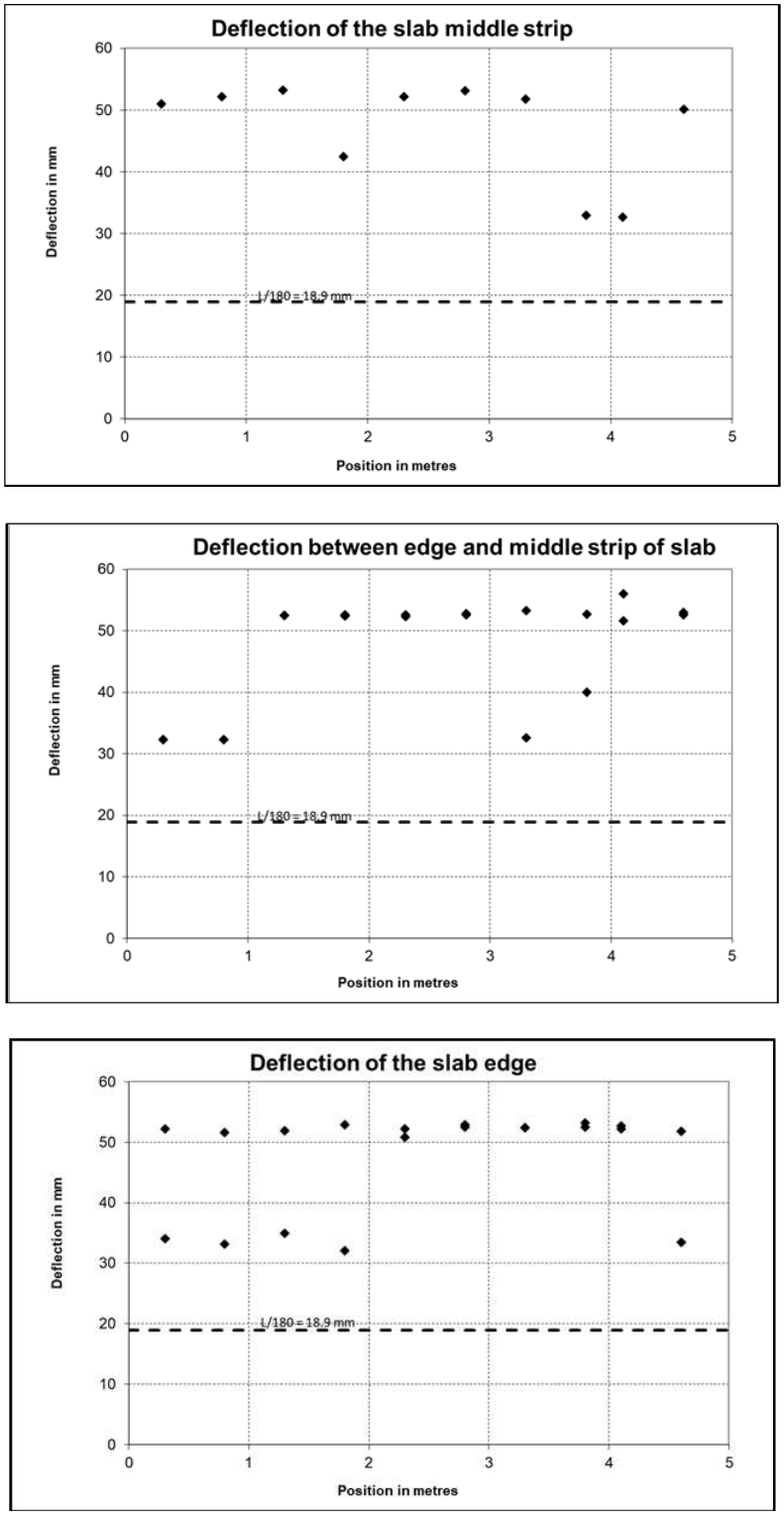


Figure 7: Deflection of the precast slab panel centreline, next to precast slab panel edges and the precast slab panel edges

4.2 Deflection Recovery

Deflection recovery after removal of superimposed load are summarised in Tables 3 and 4.

Precast slab panel centreline: The deflection recovery of the precast slab panel centreline after removal of superimposed load was a maximum of 78%, a minimum of 56% and a mean of 66%.

Precast slab panel at edges: The deflection recovery of the precast slab panel edges after removal of superimposed load was a maximum of 96%, a minimum of 68% and a mean deflection of 81%.

Precast slab panel next to edge: The deflection recovery of the precast slab panel next to edges after removal of superimposed load was a maximum of 98%, a minimum of 62% and a mean deflection of 76%.

Whole precast slab panel**:** The total deflection recovery of the slab panel after removal of superimposed load was a maximum of 98%, a minimum of 56% and a mean deflection of 76%.

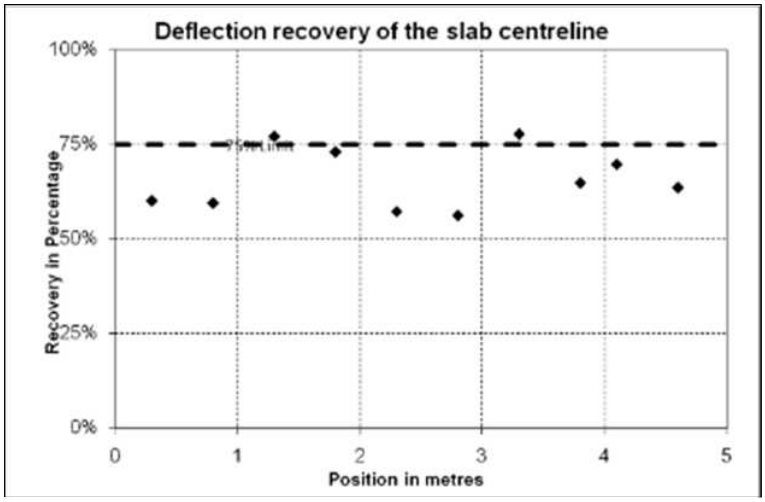
Table 3: Mean precast slab panel deflection recovery

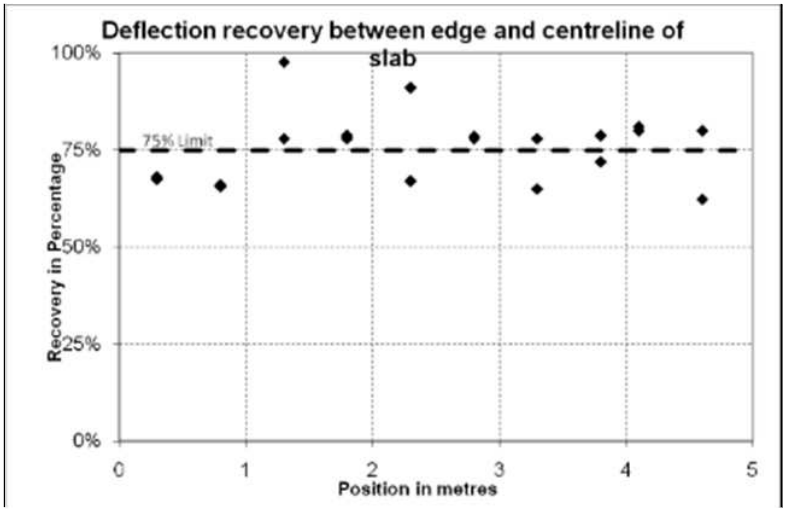
Position	Recovery	Condition	Remarks
Along edge of slab panel	81%	>75%	Above limit
Along next to edge of slab panel	76%	>75%	Above limit
Along centreline of slab panel	66%	<75%	Below limit
Average of all slab panel	76%	>75%	Above limit

Table 4: Maximum and minimum precast panel deflection recovery

Position	Minimum Recovery	Maximum Recovery
Along edge of precast slab panel	68%	96%
Along next to edge of precast slab panel	62%	98%
Along centreline of precast slab panel	56%	78%
Average of all precast slab panel	56%	98%

For the mean deflection recovery, the BS 8110 recommended limit value is to be above 75% which was achieved except for the centreline deflection recovery which was less than the recommended value. However, the average deflection recovery was higher than 75%. Generally, the average deflection recovery was within the acceptable limits and was therefore adequate. However, some individual spot levels had deflection recovery below the limiting value of 75% while the majority were above 75 % suggesting that the slab had substantial elasticity and strength. However specific areas needed attention such as along the centreline of the slab panel where the deflection recovery was lower than the limit as seen in Figure 8. Hence this zone required strengthening while the edge of the slab panel showed acceptable deflection recovery hence did not need much strengthening apart from restoration of loose concrete cover. This difference between the edge zone and the centre zone is attributed to their difference in structural depth, whereby the edge zone had higher structural depth compared to the centre zone.





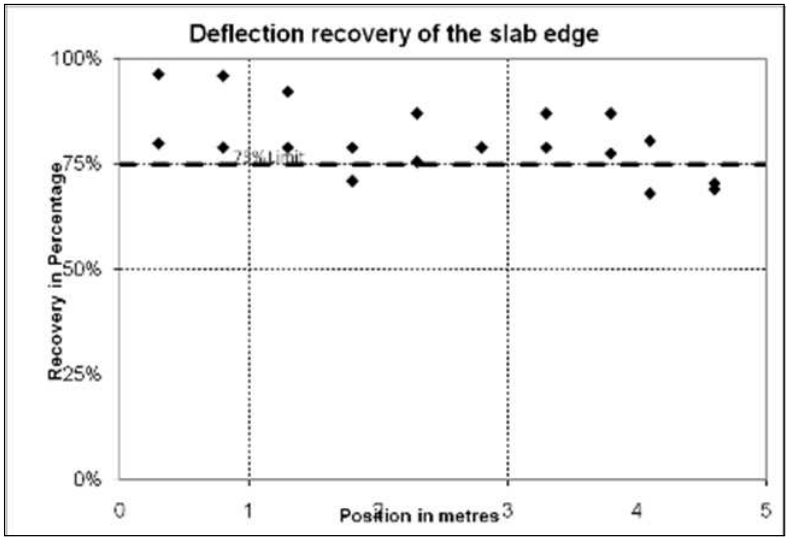


Figure 8: (a) Percentage deflection recovery of the precast slab panel centreline, next to the precast slab panel edges and the precast slab panel edges

5.0 CONCLUSION AND RECOMMENDATION

The slab has been weakened up from the fire attack as the deflections were all above the highest limiting value of L/180 for a roof slab according to ACI 318-14 and L/250 according to Eurocode 2 and BS 8110.

The results of deflection recovery of the precast reinforced concrete slab panel was clearly different between the ribs on the edges and the slab middle area.

It is concluded that analysis of results of load testing after fire accident has to consider the different structural zones in a slab instead of considering it as one entity.

The slab has to be divided into zones in terms of structural parameters such as location and dimensions which may be affected differently by fire accident. This includes whether it is on the edge or in the middle and whether there is a difference in structural dimensions such as length or thickness.

The interventions or corrective measures be applied by considering each zone separately, in which case each zone can have a different corrective measure such as extent of strengthening and acceptability.

Based on the mean deflection recovery test results value, the slab could be concluded to be generally acceptable for further use.

However, based on individual structural zones, the edge zone with ribs was acceptable while the middle strip slab zone was not acceptable hence required corrective interventions before it could be used again. The middle strip can be strengthened by introducing at its soffit a longitudinal beam of the same size as the longitudinal edge beams.

REFERENCES

American Concrete Institute, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary,

British Standards Institution. (EC2) Eurocode 2: Design of Concrete Structures, BSI, London.

British Standards Institution (BS 8110-1) Structural Use of Concrete, Code of practice for design and construction, BSI, London.

British Standards Institution (BS 8110-2) Structural Use of Concrete, Code of practice for special circumstances, BSI, London.

Casadei, P., Parretti, R., Nanni, A., and Heinze, T., (2005), In situ load testing of parking garage reinforcement concrete slabs, Comparison between 24h and cyclic load testing, Practice Periodical on Structural design and Construction, Vol. 10, Issue 1, ASCE.

Central Materials Laboratory, (2000) Laboratory Testing Manual 2000, Novaum Grafisk AS, Skjetten Norway, ISBN 9887-8891-3-1.

Hola, J., and Schabowicz, K., (2005), New technic of Nondestructive assessment of concrete strength using artificial intelligence, NDT&E International Vol. 38 251-259,

Jeremy Lucas, Thomas Causins, Micheal Brown, Stephen Sharp and Stephen Lane (2014), Structural load testing and flexural analysis of the route 701 bridge in Louisa County, Virginia, Virginia Transport Research Council, USA.

Mahadikand, A. B., Jaiswal,M. H., Structural Audit of Buildings, International Journal of Civil Engineering Research, (2014), ISSN 2278-3652 Volume 5, Number 4 (2014), pp. 411-416© Research India Publications http://www.ripublication.com/ijcer.htm

Saleem, M. A., Siddiqi, Z. A., Javed, M. A., and Aziz, M. (2012), Nondestructive evaluation of an existing concrete structure using load test and core test, Pakistani Journal of Engineering and Applied Sciences, Vol 11 page 66-72.

Samarin, A., and Meynink, P., (1981), Use of combined ultrasonic and rebound hammer method for determining strength of concrete structural members, Concrete International, Vol 3 (3) 25-29.