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# Seismic evaluation of existing reinforced concrete bridges in Karachi (Pakistan) with lightweight concrete in deck slab

**Abstract:** Bridges are lifeline structures and vulnerable to significant damage particularly in earthquake prone region like Karachi. The paper evaluated the seismic response of existing prototype reinforced concrete (RC) bridges in Karachi as a function of their economic and commercial importance. Three case study bridges with different configurations in sub and super-structures were identified from authorized stakeholders and evaluated for overall response with and without lightweight concrete (LWC) in deck slab. A detailed nonlinear finite element model of each case-study bridge has been developed and analyzed through nonlinear static pushover analysis. The behavioral changes were investigated in terms of engineering demand parameters including base shear and displacement in bridge pier along with fragility functions for both the cases. Results showed that seismic demand in terms of base shear and displacement in bridge pier curtailed in case of LWC as compared to normal weight concrete (NWC). The average reduction in base shear is 7% and in displacement it is around 6%. Furthermore, damage capacities obtained from fragility functions for different limit states increased from minor to collapse limit states in LWC as compared to counterpart. This increment 2-7% in Bridge-I, 6-12% in Bridge-II and 3-6% in Bridge-III were observed from minor to collapse damage states.

**Keywords:** damage matrix, fragility functions, lightweight concrete LWC, pushover analysis, seismic phenomena, vulnerability.

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# 1. INTRODUCTION

In Pakistan, Karachi City, which contributes 20% to Pakistan's GDP and 30% to industrial production, has significant capital investment in the road transportation network, including bridges. Karachi underwent rapid urban development, creating four signal-free corridors, over twenty bridges, and three underpasses [1]. Eighteen towns exist in Karachi and five cantonment areas; these towns are connected by major roads, railway lines, seaports and airports, as shown in Fig. 1. 224 bridges and flyovers interconnect the road network; Table 1 illustrates the characteristics of bridges in each town and cantonment area [2]. The 2005 earthquake in Kashmir caused extensive destruction, seismic vulnerability of the country was discerned, and Karachi, which is prone to seismic disturbances because of being located at the convergence of the Eurasian, Arabian, and Indian tectonic plates, requires extra vigilance in existing and newly designed infrastructure within the city [2-4]. Considering the seismic susceptibility of the town, it is crucial to obtain the probabilistic intensity of ground motions and their effect on the structures. As reported in the literature, ignorance in the past has led to fatal consequences for the I-95 Mianus River Bridge, Schoharie Creek Bridge and Cypress Viaduct [5]. According to the West Pakistan Highway Code (WPHC), the basic design standard considered in Pakistan for road infrastructure lacks design provisions such as seismic design forces up to 2-6 % of dead load and is inconsistent with regionally recently estimated and updated PGA [6]. At the same time, it is suggested from observations that several cities of Pakistan are categorized into significant ground acceleration regions and can undeniably undergo higher PGAs.

Along with these insufficiencies in WPHC, it has become obsolete compared to the American Association of State Highway and Transportation Officials (AASHTO, 2007) [7] and the Federal Highway Administration (FHWA, 2006) [8]. Although there are various works of literature available for the above deficiencies like Chaudhry Muhammad Tariq Amin, 1996, Qureshi DSM, 1996, and Syed M. Ali, 2011, the need for a new radical and significant research in reducing seismic induction in the infrastructure of Karachi is the ultimate demand of the situation [2, 9-10]. This demand for society raises the need for comprehensive study in the domain of LWC in infrastructure and particularly in bridges due to differences in weight. However, it is crucial to understand better the technicalities being implemented in the current world of designing with LWC, which enjoins the vision of this study to provide a certain degree of knowledge with the help of comparative analyses.

Table 1. Bridges and flyover in Karachi

| Town | Flyover | Bridge | Total |
|---|---|---|---|
| Baldia | 0 | 2 | 2 |
| Bin Qasim | 3 | 2 | 5 |
| Gadap | 2 | 22 | 24 |
| Gulberg | 8 | 5 | 13 |
| Gulshan-e-Iqbal | 16 | 5 | 21 |
| Jamshed | 5 | 9 | 14 |
| Keamari | 7 | 13 | 20 |
| Korangi | 2 | 3 | 5 |
| Landhi | 0 | 2 | 2 |
| Liaquatabad | 7 | 13 | 20 |
| Lyari | 2 | 0 | 2 |
| Malir | 1 | 4 | 5 |
| New Karachi | 0 | 7 | 7 |
| North Nazimabad | 4 | 8 | 12 |
| Orangi | 1 | 20 | 21 |
| Sindh Industrial and Trading State | 3 | 13 | 16 |
| Saddar | 9 | 0 | 9 |
| Shah Faisal | 1 | 1 | 2 |
| Total | 71 | 129 | 200 |
| Contonment Areas | | | |
| Clifton Cantonment | 2 | 6 | 8 |
| Faisal Cantonment | 8 | 2 | 10 |
| Karachi Cantonment | 3 | 0 | 3 |
| Korangi Cantonment | 0 | 1 | 1 |
| Malir Cantonment | 2 | 0 | 2 |
| Total | 15 | 9 | 24 |

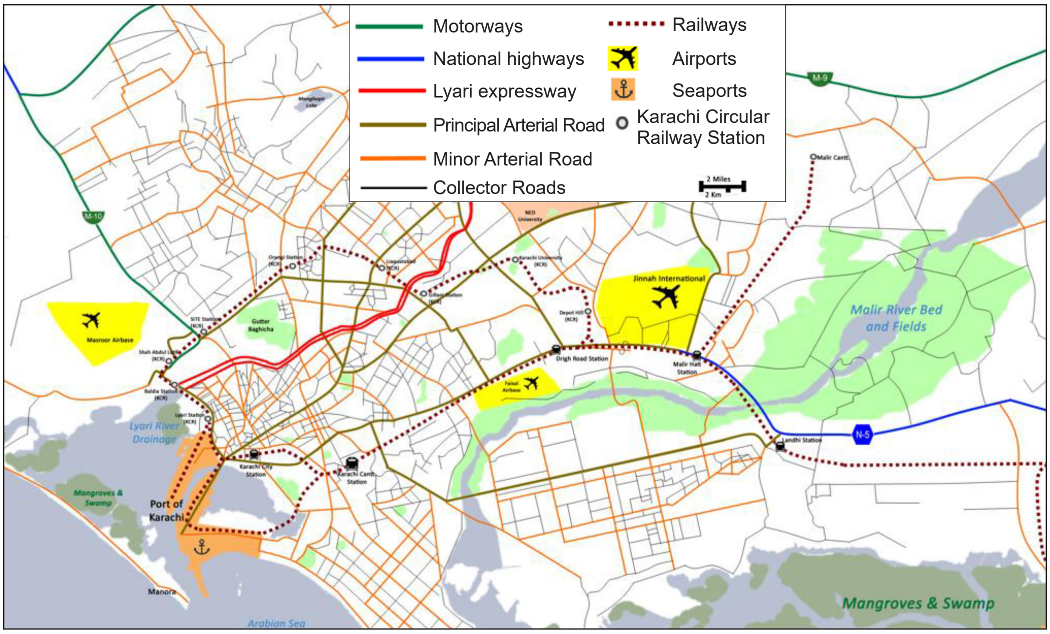


Fig. 1. Major roads, railway lines, ports and airports in Karachi

## 2. RESEARCH SIGNIFICANCE

LWC, having low density, provides lesser weight than NWC. Since seismic actions are directly related to the weight of the structures, this facilitates reducing the vulnerability against seismic actions to an extent [11]. Therefore, the main focus of the current study is to quantify the reduction in vulnerability by using LWC, particularly in super-structure elements, compared to the counterpart NWC.

## 3. CASE STUDY BRIDGES

### 3.1. MATERIAL MODEL

To carry out the comparative analyses of the bridges with two different materials in the deck slab, a different inventory of bridges was required, having diverse information and characteristics. In the current study, a total of three bridge cases are considered in the analysis part of major arterial traffic lanes of Karachi (Fig. 2), which are assorted as Stadium Road bridge (Bridge-I in Central District), Malir-15 bridge (Bridge-II in Malir District), and Naagan flyover (Bridge-III in North Karachi-District).

As mentioned, the main emphasis of this study is on material change along with different assessments. Therefore, the shift from NWC to LWC was only considered on the super-structure level. i.e., deck slab. Considering the change in the deck slab, it is essential to study the post-analysis behaviour of the structure to idealize the material's effectiveness. While being a flexure member, according to Mohammad Zareh, moment capacity would reach up to 92% in LWC as that of NWC, but approximately 40%

Fig. 2. Existing identified case-study bridge: a) bridge I, b) bridge II, c) bridge III

more deflection would be encountered [12]. Hence, based on available literature the details of material properties considered in analyses for NWC are shown in Table 2 and for LWC are presented in Table 3. In contrast, Fig. 3 shows the stress-strain curves of unconfined plots of both the concrete.

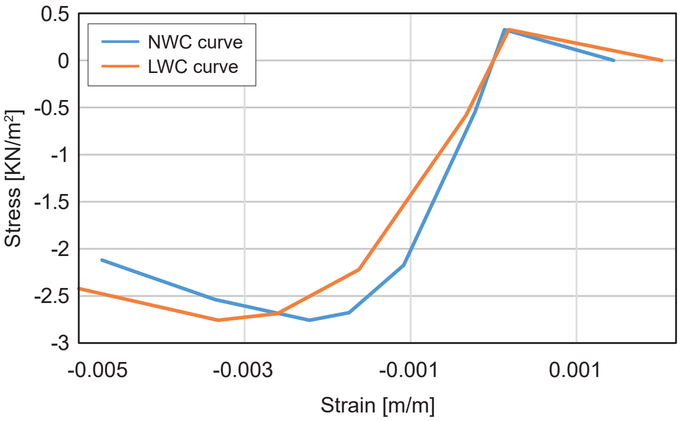


Fig. 3. Stress-strain curve for concrete (Mander's model), NWC and LWC (unconfined)

## 3.2. LOADS AND LOAD COMBINATION

Dead loads include the self-weight of the bridge, the weight of wearing surfaces, and utilities. Whereas live loads are moving loads, they include vehicular and pedestrian traffic. The considered live load in this study is Class "A" Truck Loading given by WPHC (West Pakistan Highway Code), shown in Fig. 4.

The load combination (1) considered is strength-1 given by AASHTO LRFD:

$$Load\ combination = 1.25(DL) + 1.25(SDL) + 1.75(LL), \quad (1)$$

where:

*DL* – Dead Load
*SDL* – Super Dead Load
*LL* – Live Load

Table 2. Normal weight concrete properties used in analysis

| No. | Normal weight concrete | |
|---|---|---|
| | Linear properties | |
| 1 | Weight per unit volume $W_t$ [kN/m³] | 23.56 |
| 2 | Compressive strength $f'_c$ [MN/m²] | 27.6 |
| 3 | Poisson's ratio $U$ [–] | 0.2 |
| 4 | Coefficient of thermal expansion α [–] | 5.5E-6 |
| 5 | Elastic modulus $E$ [kN/m²] | 24.8E6 |
| 6 | Shear modulus $G$ [kN/m²] | 10.3E7 |

Table 3. Lightweight concrete properties used in analysis

| No. | Lightweight concrete | |
|---|---|---|
| | Linear properties | |
| 1 | Weight per unit volume $W_t$ [kN/m³] | 18.06 |
| 2 | Compressive strength $f'_c$ [MN/m²] | 27.6 |
| 3 | Poisson's ratio $U$ [–] | 0.2 |
| 4 | Coefficient of thermal expansion α [–] | 5.5E-6 |
| 5 | Elastic modulus $E$ [kN/m²] | 17.7E6 |
| 6 | Shear modulus $G$ [kN/m²] | 7.4E6 |
| 7 | Shear reduction factor λ [–] | 0.75 |

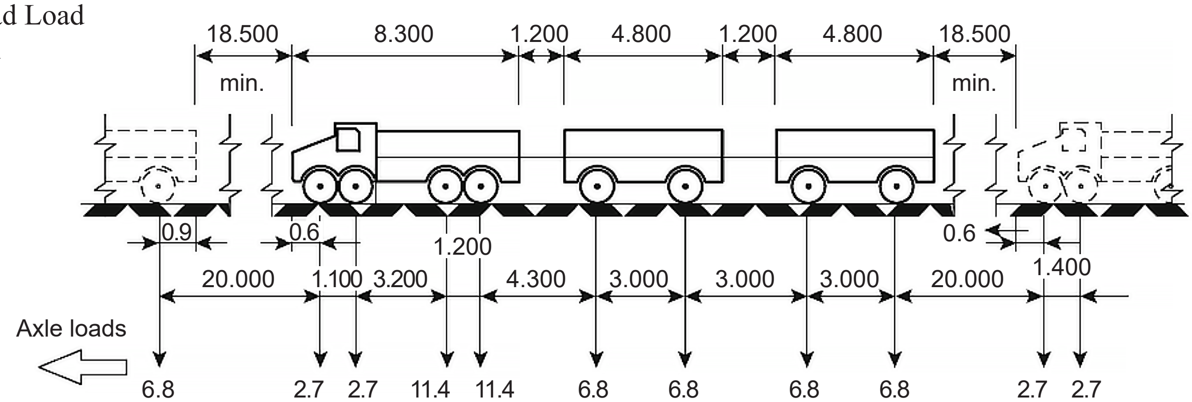


Fig. 4. Class "A" truck design (source WPHC 1997)

## 3.3. FINITE ELEMENT MODELLING

The Finite Element modelling (FEM) was carried out using SAP2000, which lowers the complexity of the bridge geometries. Line-element forms were used for Girders and Columns modelling while Deck Slab was modelled as area-element. Connections between different structural elements or the joints and offsets were assumed to be typical as integrated into SAP2000. Abutments were modelled as external hinges due to their respective details. Furthermore, for Karachi, the soil profile and seismic zone factors were considered SD and Zone 2B, as reported in BCP-2007 [13]. All three bridges are essential and have a single pier per transom with importance factor $I$ as unity and response modification factor $R$ as 2. In Tables 4-6 shown all the structural properties and details of the bridges modelled and analyzed. It should be known that bridge-II has two similar sides. Therefore, only one side was considered in the analysis. Apart from structural data, all other superimposed elements, including curbs, road carpets, and other utilities, were applied as linear and area loads (Fig. 5).

Table 4. Structural properties and details of Bridge-I

| Components of bridges | Description | Components of bridges | Description |
|---|---|---|---|
| Horizontal layout | None | Column-foundation support cond. | Fixed |
| Vertical layout | Yes. Change in grade at each station | Bearing | Translation (fixed) rotation (free) |
| Number of girders | 8 | Abutment (girder support condition) | Pin/hinge |
| Deck slab width [m] | 16.65 | Cap beam section [m] | Transom ($d$ = 2, $w$ = 2) |
| Slab thickness [mm] | 300 | Cap beam length [m] | 16.650 |
| Diaphragm thickness [mm] | 450 | Prestress type | Pretensioned |
| Type of restrainer | None | No. of ducts | 4 |
| Number of lanes | 4 | No. of strands | 19 |
| Type of girder | Precast U-shape | Area of strands [mm$^2$] | 98.7 |
| Number of piers | 6 | Shortest span [m] | 27.46 |
| Number of spans | 7 | Longest span [m] | 35.035 |

Table 5. Structural properties and details of Bridge-II

| Components of bridges | Description | Components of bridges | Description |
|---|---|---|---|
| Horizontal layout | None | Column-foundation support cond. | Fixed |
| Vertical layout | Yes. Change in grade at each station | Bearing | Translation (fixed) rotation (free) |
| Number of girders | 7 | Abutment (girder support condition) | Pin/hinge |
| Deck slab width [m] | 8.7 | Cap beam section [m] | Transom-1 ($d$ = 0.7, $w$ = 3.7)<br>Transom-2 ($d$ = 0.7, $w$ = 2.68) |
| Slab thickness [mm] | 240 | Cap beam length [m] | 8.7 |
| Diaphragm thickness [mm] | 400 | Prestress type | Pretensioned |
| Type of restrainer | None | No. of ducts | 3 |
| Number of lanes | 2 | No. of strands | 12 |
| Type of girder | Precast I-shape | Area of strands [mm$^2$] | 92.9 |
| Number of piers | 8 | Shortest span [m] | 26 |
| Number of spans | 9 | Longest span [m] | 35 |

Table 6. Structural properties and details of Bridge-III

| Components of bridges | Description | Components of bridges | Description |
|---|---|---|---|
| Horizontal layout | None | Column-foundation support cond. | Fixed |
| Vertical layout | Yes. Change in grade at each station | Bearing | Translation (fixed) rotation (free) |
| Number of girders | 4 | Abutment (girder support condition) | Pin/hinge |
| Deck slab width [m] | 9.57 | Cap beam section [m] | Transom ($d = 2$, $w = 1.936$) |
| Slab thickness [mm] | 175 | Cap beam length [m] | 9.57 |
| Diaphragm thickness [mm] | 300 | Prestress type | Pretensioned |
| Type of restrainer | None | No. of ducts | 5 |
| Number of lanes | 4 | No. of strands | 19 |
| Type of girder | Precast U-shape | Area of strands [mm$^2$] | 98.7 |
| Number of piers | 17 | Shortest span [m] | 20 |
| Number of spans | 18 | Longest span [m] | 35 |

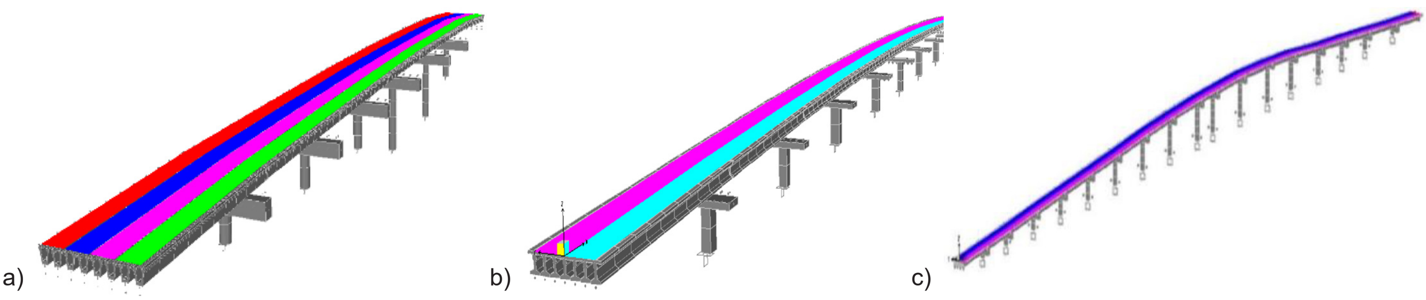


Fig. 5. FEM modelled bridge cases: a) Bridge-I, b) Bridge-II, c) Bridge-III

For seismic evaluation of existing identified bridges, the demand spectra shown in Fig. 6 were imposed on case-studies bridges obtained from AASHTO 2007 corresponding to the spectral accelerations for a short period $S_{DS} = 0.5$ and for 1.0 sec period $S_{D1} = 0.2$ [14-15].

Pushover analysis offers reasonable estimates of global and local inelastic deformation demands for structures that vibrate primarily in the fundamental mode. Geometrical nonlinearity, material inelasticity, and internal force redistribution are entirely considered in the analysis [16]. In pushover analysis, the amplitude of the lateral load is gradually increased while preserving a preset concentrated load at the structure's height in which the sequence of cracking, plastic hinging, and structural component failure is monitored.

At potential cracking or plastic zones under lateral strain, each pier was simulated with a flexural (P-M2-M3) hinge at a distance of 0.9$H$ ($H$ being the dock's height from the transom to the top of the pile cap). To account for the failure of the pier before super-structure elements and to capture the material nonlinearity, a plastic hinge (P-M2-M3 hinge) was assigned on each bridge's pier, as shown in Fig. 7.

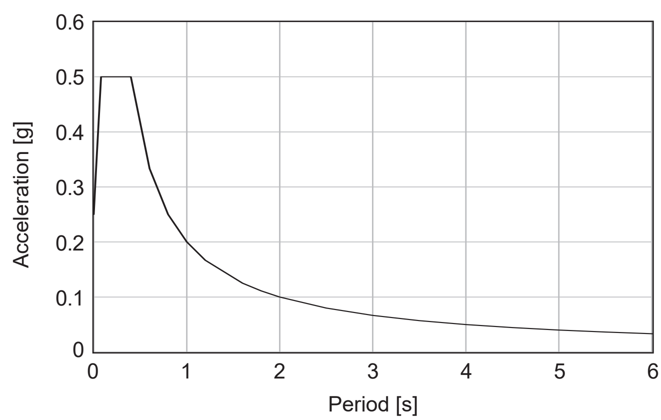


Fig. 6. Code specified site-specific response spectra

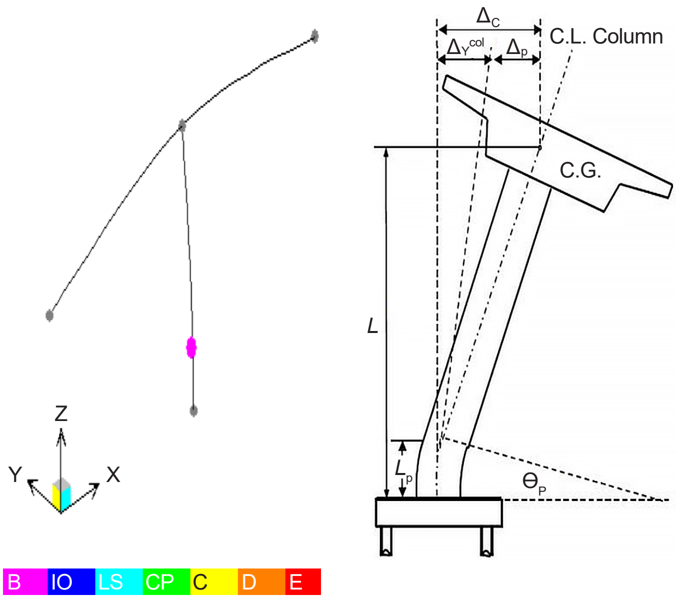


Fig. 7. P-M2-M3 flexural hinge on the pier at 0.9*H* in SAP2000

A top displacement vs base-shear obtained from the displacement-controlled option available in the program for every pier was generated, which is conventionally called the pushover or capacity curve, as shown in Fig. 8. Later, the Capacity Spectrum Method (CSM) of ATC 40 [17] uses the pushover curve to calculate the global displacement demand also known as target displacement for the individual pier in all the case-study bridges shown in Table 7-9. In the Capacity Spectrum Method, it is assumed that a nonlinear SDOF system's maximum inelastic deformation can be estimated using the maximum deformation of a linear elastic SDOF system with an analogous period and damping of 5%. As a function of the effective damping ratio $\beta_{eq}$, ATC 40 provides reduction factors to reduce spectral ordinates in the constant acceleration and constant velocity regions. The spectral reduction factors are calculated by Eq. (2-3):

$$SR_A = \frac{3.21 - 0.68 \cdot ln\ (100 \cdot \beta_{eq})}{2.12}, \tag{2}$$

$$SR_V = \frac{2.31 - 0.41 \cdot ln\ (100 \cdot \beta_{eq})}{1.65}. \tag{3}$$

The equivalent effective damping ratio $\beta_{eq}$ can be calculated by Eq. (4):

$$\beta_{eq} = K\beta_o + 5\%, \tag{4}$$

where:

$K$ – factor depends on the structure behavior type named as A, B and C; the value of $K$: for type A is 1.0, type B is 0.67 and type C is 0.33 as per ATC40,

$\beta_o$ – hysteretic damping determined by Chopra 1995 [18],

5% – initial viscous damping ratio.

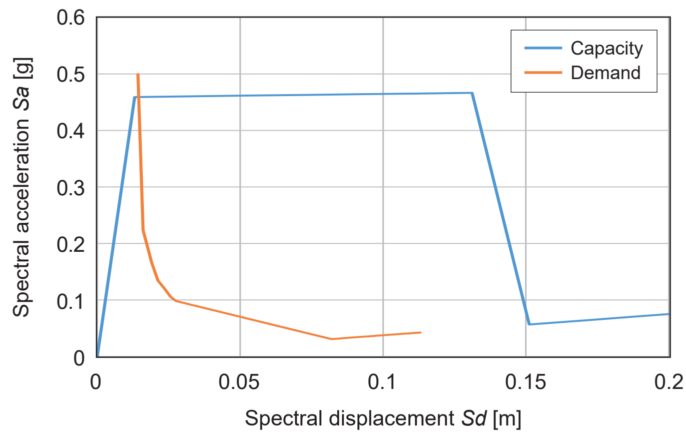


Fig. 8. Capacity-demand curves for a single pier

Table 7. Performance point using capacity demand spectra of Bridge-I

| Piers No. | Normal weight concrete-NWC | | Lightweight concrete-LWC | |
|---|---|---|---|---|
| | Force [kN] | Displacement [mm] | Force [kN] | Displacement [mm] |
| 1 | 5121 | 3.1 | 4908 | 3.2 |
| 2 | 4890 | 6.8 | 3742 | 5.5 |
| 3 | 4492 | 11.9 | 3602 | 10.7 |
| 4 | 4124 | 34.6 | 3849 | 29.8 |
| 5 | 4152 | 49.1 | 4001 | 49 |
| 6 | 4632 | 21.7 | 4002 | 19.4 |

Table 8. Performance point using capacity demand spectra of Bridge-II

| Piers No. | Normal weight concrete-NWC | | Lightweight concrete-LWC | |
|---|---|---|---|---|
| | Force [kN] | Displacement [mm] | Force [kN] | Displacement [mm] |
| 1 | 4914 | 9.5 | 4623 | 8.2 |
| 2 | 4925 | 14.2 | 4742 | 13.7 |
| 3 | 5465 | 8.5 | 5151 | 8.2 |
| 4 | 5051 | 15 | 4905 | 14.6 |
| 5 | 3945 | 19 | 3813 | 18.0 |
| 6 | 5785 | 12 | 5601 | 11.5 |
| 7 | 5966 | 11.5 | 5770 | 11.2 |
| 8 | 4388 | 11.7 | 4143 | 11.3 |

Table 9. Performance point using capacity demand spectra of Bridge-III

| Piers No. | Normal weight concrete-NWC | | Lightweight concrete-LWC | |
|---|---|---|---|---|
| | Force [kN] | Displacement [mm] | Force [kN] | Displacement [mm] |
| 1 | 2762 | 0.153 | 2633 | 0.149 |
| 2 | 3548 | 0.471 | 3444 | 0.457 |
| 3 | 3842 | 0.509 | 3728 | 0.494 |
| 4 | 3836 | 0.507 | 3727 | 0.492 |
| 5 | 3831 | 0.506 | 3718 | 0.491 |
| 6 | 3829 | 0.505 | 3716 | 0.49 |
| 7 | 3828 | 0.504 | 3715 | 0.489 |
| 8 | 3826 | 0.503 | 3712 | 0.488 |
| 9 | 3042 | 0.400 | 2955 | 0.389 |
| 10 | 2896 | 0.373 | 2815 | 0.362 |
| 11 | 4237 | 0.477 | 4110 | 0.433 |
| 12 | 4113 | 0.463 | 4030 | 0.423 |
| 13 | 3909 | 0.477 | 3793 | 0.387 |
| 14 | 4036 | 0.439 | 3916 | 0.425 |
| 15 | 3740 | 0.400 | 3629 | 0.388 |
| 16 | 3990 | 0.427 | 3874 | 0.415 |
| 17 | 4208 | 0.475 | 4080 | 0.441 |

## 3.4. DAMAGE STATES

To evaluate the probability of damage in bridges with NWC and LWC due to different ground motions, analytical fragility curves were generated in terms of intensity measures PGA using the Lagomasino and Giovinazzo approach [19]. As reported in the literature, five damage states generally exist based on yield $D_y$ and ultimate displacement $D_u$ capacities when members reach their inelastic deformation zone.

At damage state "None" (DS-1), the bridge is in a pre-yield state and suffers no loss, while at damage state "Minor" (DS-2), cracking and spalling of the concrete occurs and requires surface repair. In the damage state, "Moderate" (DS-3), bar buckling is implied, along with spalling. It involves reparation of the bridge elements, while at damage state "major" (DS-4), fracturing and degradations occur where the components are supposed to be rebuilt. At the final damage state "Collapse" (DS-5), reconstruction or retrofitting of the entire structure is required. To develop the fragility functions, HAZUS MR-4 methodology has been adopted herein; in this approach, the intensity measures (equivalent PGAs) calculated corresponding to different damage states proposed by Lagomasino and Giovinazzo's in terms of displacements such as ($0.7Dy$) for "DS-2", ($1.5D_y$) for "DS-3", [$0.5(D_y + D_u)$] for "DS-4", and $D_u$ for "DS-5" [9, 19-20].

## 4. RESULTS AND DISCUSSION

Initially, the engineering demand parameters (EDPs), base shear, and displacement were calculated for both the cases (NWC and LWC) at the performance point of the individual pier. Then, a reduction in EDPs was estimated. Results obtained are presented in terms of average reduction in base shear and displacement demand for all the case-study bridges. From LWC in the deck slab for Bridge-I, an average decrease of 12% in base shear and 8.5% in displacement demand was observed. Similarly, for Bridge-II, a reduction of 4.5% in base shear and 5% in displacement demand, whereas, for the case study Bridge-III, a decrease of 3% in base shear and 5% in displacement demand are observed.

Furthermore, the average intensity measures (IMs) PGA(g) have been calculated for each damage state from NWC and LWC in the deck slab (Table 10). Finally, fragility functions have been developed for different damage states and depict the analytical variation occurring from NWC to LWC employed in the deck slab (Fig. 9). Results obtained from fragility functions show that the seismic capacity of investigated bridges with LWC increases at different damage states. This increment of 2-7% in Bridge-I, 6-12% in Bridge-II and 3-6% in Bridge-III were observed from minor to collapse damage states. Also, it has been noticed that this increment is directly proportional to the decrease in seismic weight associated with super-structure elements.

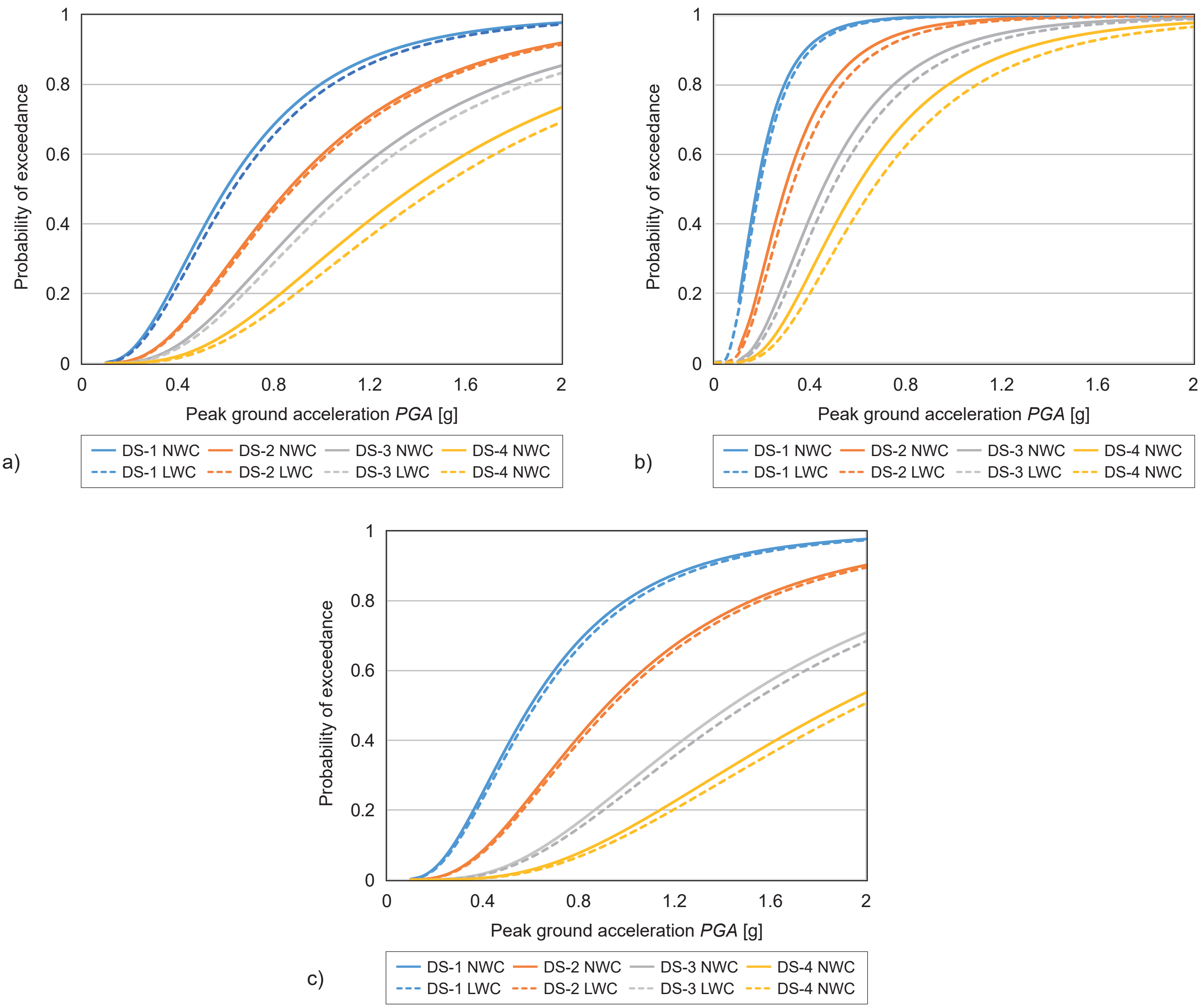


Fig. 9. Fragility curves for NWC and LWC in case: a) Bridge-I, b) Bridge-II, c) Bridge-III

## 5. CONCLUSIONS AND RECOMMENDATIONS

The current study investigates the seismic risk associated with lifeline structures, particularly bridges in seismically active regions and the largest metropolis of Pakistan. To this end, three existing RC bridges identified in Karachi and their computational models developed on FEM code analyzed for gravity and lateral loads by employing linear and nonlinear static analyses. Results were presented regarding variations in performance points due to material changes in the deck slab. Furthermore, fragility functions were considered to obtain the damages in respective bridges as part of the global analysis procedure. The essential conclusions drawn from the conducted study are provided in progressive order below:

1. The nonlinear static analysis encompasses the significant reduction observed in EDP's seismic base shear and displacement demand by the change from NWC to LWC in the deck slab. The average reduction in base shear is 7%, and it is around 6% in displacement.

2. The seismic risk quantification presented herein in terms of fragility functions through analytical PGAs obtained from a series of pushover analyses corresponding to different limit states such as DS-1, DS-2, DS-3 and DS-4. It was found that a significant increment was observed in damage capacities. This increment of 2-7% in Bridge-I, 6-12% in Bridge-II and 3-6% in Bridge-III were observed from minor to collapse damage states.

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## Ocena stanu mostów żelbetowych z płytą pomostową z betonu lekkiego użytkowanych na terenie Karachi w Pakistanie z uwzględnieniem występujących tam zjawisk sejsmicznych

**Streszczenie:** Mosty to konstrukcje o długotrwałym okresie eksploatacji, które w regionach takich jak Karaczi w Pakistanie narażone są na znaczące uszkodzenia w wyniku występujących tam trzęsień ziemi. W artykule oceniono odporność na zjawiska sejsmiczne aktualnie użytkowanych na terenie ww. aglomeracji konstrukcji mostowych wykonanych z żelbetu RC (ang. *reinforced concrete*) w kontekście ich znaczenia ekonomicznego. Na podstawie decyzji ich zarządców wytypowano do badań trzy obiekty mostowe o różnych konstrukcjach posadowienia oraz odmiennej konstrukcji płyty pomostowej: wykonanej w części z betonu lekkiego LWC (ang. *lightweight concrete*) lub w całości z betonu zwykłego NWC (ang. *normal weight concrete*). Opracowano model MES każdej konstrukcji, a następnie w celu wykazania przydatności wbudowania betonu lekkiego (LWC) w płytę pomostu, przeprowadzono analizę w zakresie nieliniowym aż do zniszczenia konstrukcji (ang. *pushover analysis*). Wpływ oddziaływań sejsmicznych na trwałość konstrukcji mostowych z płytą pomostową LWC bądź NWC, zbadano pod kątem określenia wymaganych wartości parametrów wytrzymałościowych na ścinanie, przemieszczenie i pękanie filarów. Wyniki badań wykazały, że w przypadku zastosowania w płycie pomostowej - betonu lekkiego LWC, odporność konstrukcji na obciążenia sejsmiczne jest większa w stosunku do wytrzymałości obiektów z pomostem wykonanym w całości z betonu konwencjonalnego NWC. Wykazano, że wytrzymałość na ścinanie zmniejszyła się średnio o około 7%, a przemieszczenie podpór o około 6%. Ponadto, odporność na kruche pęknięcia, określona dla różnych stanów granicznych uszkodzeń, była większa dla mostu z płytą LWC w porównaniu z konstrukcją, w której pomost wykonany był w całości z betonu NWC. Ten wzrost odporności wynosił odpowiednio: 2-7% – w przypadku Mostu-I, 6-12% – w przypadku Mostu-II oraz 3-6% – w przypadku Mostu-III.

**Słowa kluczowe:** beton lekki LWC, funkcje kruchości, macierz uszkodzeń, nieliniowa analiza statyczna, podatność, zjawiska sejsmiczne.