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Catalytic Removal of Ozone by Pd/ACFs and Optimal Design of Ozone Converter for Air Purification in Aircraft Cabin

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Abstract
Ozone in aircraft cabin can bring obvious adverse impact on indoor air quality and occupant health. The objective of this study is to experimentally explore the ozone removal performance of flat-type catalyst film by loading nanometer palladium on the activated carbon fibers (Pd/ACFs), and optimize the configuration of ozone converter to make it meet the design requirements. A one-through ozone removal unit with three different Pd/ACFs space was used to test the ozone removal performance and the flow resistance characteristic under various temperature and flow velocity. The results show that the ozone removal rate of the ozone removal unit with the Pd/ACFs space of 1.5 mm can reach 99% and the maximum pressure drop is only 1.9 kPa at the reaction temperature of 200℃. The relationship between pressure drop and flow velocity in the ozone removal unit has a good fit to the Darcy-Forchheimer model. A new ozone converter with flat-type reactor was designed and processed based on the one-through ozone removal experiment, its ozone removal rate and maximum pressure drop were 97% and 7.51 kPa, separately, with the condition of 150℃ and 10.63 m/s. It can meet the design requirements of ozone converter for air purification and develop a healthier aircraft cabin environment.

Keywords: Ozone; Aircraft Cabin; Optimum Arrangement; Pd/ACFs; Ozone Converter.

1. Introduction
The outdoor air pollution and ventilation system pollution are two major factors influencing the indoor air quality [1]. As a special indoor environment, the air quality in the aircraft cabin is more associated with the ambient air conditions and regulated by the supplied outside air [2, 3]. Since the energy crisis of 1970s, commercial airplanes routinely cruise in the upper troposphere or the lower stratosphere where the ozone concentration can reach the level of hundreds of parts per billion (ppb), ozone will enter aircraft cabin with the bleeding air through engine compressors [4]. Ever since, more and more passengers started to complain the poor air quality caused by the ozone [5]. As to short-term exposure, studies have strengthened the evidence that exposure to the over-standard ozone concentration (>0.1ppm) will increase the mortality and respiratory morbidity rates [6]. Due to the strong oxidizing, ozone can react with the passenger’s skin oils and the leather seats in aircraft cabin, which become the important source of volatile organic compounds (VOCs) [7]. Due to the hazards of ozone, the World Health Organization had updated the air quality guideline for indoor ozone that the maximum average concentration cannot exceed 0.09 ppm in 8-hr when people exposure to the ozone environment. The Occupational Safety and Health Administration (OSHA, the United States of American) also required the maximum ozone concentration of 0.1 ppm when human exposure to such environment for

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The U.S. Federal Aviation Administration (FAA) regulations require that the ozone concentration in aircraft cabin cannot exceed 0.1 ppm when flying time exceeds 4-hr. Chinese ‘indoor air quality standards’ (GB/T 18883-2002) requires that the indoor ozone concentration cannot exceed 0.08 ppm [10].

For meeting the above air quality standards, some types of airplanes have been equipped with ozone converter in the environmental control system (ECS) to reduce the ozone concentration [10, 11]. According to the data released from European aircraft manufacturer Airbus and Boing Company, the ozone converter is a mandatory device on the long flight commercial aircrafts like A320, A330 and B767, B777, while the rest thousands of short flight airplanes were installed optionally [12]. At present, many aircrafts do not have the air-cleaning facility to remove ozone, so the mean ozone level exceeding 0.1 ppm over the course of flight often occurs. It is obvious that the further technological research and development in the field of ozone conversion is critical to improve the air quality in aircraft cabin [13, 14].

![Flowchart of Research Methodology](image)

Figure 1. The flowchart of research methodology

Researchers have proposed various ozone removal techniques for ozone removal, such as adsorption [15], thermal decomposition [16], electromagnetic radiation decomposition and catalytic decomposition [17, 18]. Because of the efficiency and economy, catalytic decomposition of ozone is an ideal technique for indoor air purification [19]. Catalytic decomposition consist of thermal-catalytic decomposition and photocatalytic decomposition [20-22]. Lu et.al has experimentally confirmed that the flow velocity of the bleeding air supply to the aircraft cabin is too high to apply photocatalytic technique into the ozone removal in aircraft [10]. Thermal-catalytic decomposition is the more suitable example, because the temperature of bleeding air in the aircraft is closed to 90-200°C which can meet the...
temperature requirement for the thermal-catalytic technology [8]. The results in our previous research show that thermal catalytic by coating nanoparticle palladium on the surface of activated carbon fibers (Pd/ACFs) can remove ozone efficiently and ozone removal rate is proportional to flow resistance [23]. Through extensive literature review, very few researches focus on the design of ozone converter and test ozone removal performance under actual working condition of bleeding air from aircraft. It is necessary to focus on the optimization of reactor configuration to reduce its pressure drop and maintain ozone removal efficiency. Figure 1 shows the flowchart of research methodology. The results in this paper can provide the design basis for the application of ozone converter in environment control system of commercial aircraft [24-26].

2. Experiment Details

2.1. Experimental Apparatus

The experimental system in this paper, which can be used to adjust various parameters, such as initial ozone concentration, flow velocity and experimental temperature. The schematic experimental system is shown in Figure 2.

Figure 2. The schematic diagram of the experimental system

An air compressor drove a zero air generator to produce high pressure zero air. A dynamic gas calibrator (Thermo Environmental Inc. Model 146C) was operated with the zero air generator to control the mass flow rate of the zero gas. The ozone generator equipped with three ultraviolet lamps with a primary wavelength of 254 nm was used to introduce ozone into the experimental system. Before the experiment, the air stream containing ozone flow through the by-pass Route I. When the flow rate and ozone concentration reach steady for one hour, ozone was introduced into the reactor that was placed in the center of the electric resistance furnace.

The reactor (Figure 3) was composed of a transparent quartz glass tube with the diameter of 14mm and the length of 100 mm, an ozone removal unit was placed inside it. The ozone removal unit was made of the horizontally arranged catalyst Pd/ACFs that was prepared by coating nanometer palladium on the surface of activated carbon fibers with the same process as shown in literature [23]. The performance of ozone removal unit with different Pd/ACFs space was tested.

Figure 3. The reactor schematic diagram
Figure 4 shows the ozone removal unit with the different Pd/ACFs space. In order to meet the reactor dimensions, a Pd/ACFs fixing device with 60mm in length, 9mm in width and 10mm in height was prepared, as shown in Figure 4(a). The fixing device, 9mm in width and 2mm in height fixed the Pd/ACFs films with the dimension of 70mm in length, horizontally. For example, two flat-type Pd/ACFs films were fixed by the space of 5mm in Figure 4(b), while three and four flat-type Pd/ACFs films were fixed with pitch distance of 1.5mm and 0.3mm (measured by the vernier caliper), as shown in Figure 4(c) and (d), respectively. Due to the fixing device has a little effect on the flow process, so it is represented by the dash line, as shown in the Figure 4 (b-d).

3. Results and Discussions

3.1. The Performance of Ozone Removal Unit

3.1.1. Ozone Removal

Figure 5 shows the variation of the outlet ozone concentration with time for the three types of ozone removal units at the flow velocity of 0.3 m/s. At the time of 0 min, ozone flowed into the by-pass Route I (in Figure 2) with the initial concentration of 1.79 ppm. After the ozone concentration reached steady for one hour, it was introduced into the reactor. Here, only the steady data of the first 40 min were indicated in the Figure 4. The reaction temperature was increased with the rise rate of 40℃ every 40 min from 40℃ until it reaches 200℃. One can see that the ozone concentration decreased quickly when the reaction temperature was increased. It also decreased sharply by the increasing reaction area. Ozone concentration dropped rapidly to the level of 0.22ppm and 0.08ppm at room temperature with 3 and 4 flat-type Pd/ACFs films, respectively, while it was almost 0.99 ppm with 2 flat-type Pd/ACFs films. However, the effect of reaction area on the ozone removal is not obvious with the rise in reaction temperature. For example, the outlet ozone concentration for all three types ozone removal unit were almost zero at the temperature of 200℃. So the ozone removal over Pd/ACFs is dominated by the reaction temperature because the catalyst has the higher activity at the higher temperature.

The above results show that the ozone removal performance of the Pd/ACFs can be enhanced by increasing reaction area at lower temperature; otherwise the reaction temperature should be improved. Poshin et al. [27] evaluate the ozone removal over activated carbon filters at the ppb level of ozone concentration and found that the ozone removal capacity
can be increased by increasing the contact surface area. The similar results were founded in this study. On the other hand, the more Pd/ACFs layers lead to the decreasing space of adjacent catalyst Pd/ACFs, which shorten the flow area of ozone and increase the flow velocity on the Pd/ACFs surface. Therefore, the ozone concentration boundary layer on the surface of Pd/ACFs become thin and the more ozone can be absorbed and decomposed.

Figure 6 shows the ozone conversion rate of the ozone removal unit with the different layers of Pd/ACFs film varies with temperature. The ozone conversion rate was calculated by 100(C_in-C_out)/C_in %, where C_in and C_out is the inlet and outlet ozone concentration (ppm), respectively. The ozone conversion rate with three and four layers of catalyst Pd/ACFs was more than 90% at 40°C and further increased to 99% when the temperature reached at 200°C. However, for the two layers of Pd/ACFs, it was only 45% at room temperature, but over 97% when the temperature rose to 160°C and up to 99% at the temperature of 200°C. The results further illustrated that enhancing temperature was important for catalytic removal of ozone with the catalyst Pd/ACFs.

![Figure 5. Variation of the outlet ozone concentration with time at different reaction temperature](image1)

![Figure 6. Variation of the ozone conversion rate with temperature using the different ozone removal units](image2)
3.1.2. The Pressure Drop of Ozone Removal Units

The pressure drop is another important parameter for ozone reactor, for reducing energy consumption the low-pressure drop is recommended. The pressure drop in this research was measured by a U-tube manometer and calculated by the altitude difference of water column in the U-tube as Equation 1.

$$\Delta P = \rho gh$$

Where $\Delta P$ is the pressure drop of import and export reactor (kPa), $\rho$ is the density of water column (kg/m$^3$), $g$ is the acceleration of gravity (m/s$^2$), $h$ is the height difference of water column in the U-tube manometer (m).

![Figure 7. Variation of the pressure drop with flow velocity for the different ozone removal units](image)

Figure 7 shows the variation of the pressure drop with flow velocity for the different ozone removal units. The pressure drop increased with the rose in flow velocity for the given ozone removal unit. It also increased with the rose in the number of catalyst layers at a fixed flow velocity. For example, the pressure drop in the ozone removal unit increased from 1.37 kPa for 2 catalyst films to 2.42 kPa for 4 catalyst layers at the flow velocity of 1.1 m/s. The reason is that the more Pd/ACFs layers filled in the reactor can lead to the smaller flow area of reactor and the higher flow velocity over the catalyst surface, which result in the higher on-way resistance. Shuai et al. [28] analyzed the pressure losses in automotive catalytic converter and found that the porosity of the packed bed has a significate impact on the pressure drop of the catalytic converter, the higher porosity of packed bed contribute to the lower pressure drop. The similar conclusion was also obtained in our study.

3.1.3. The Pressure Drop Prediction

The ozone reactor filled with catalyst Pd/ACFs is supposed as a porous media, so the porous media model can analyze the relationship between pressure drop and flow velocity. Regulski et.al studied the pressure drop in the porous media by numerical and experiment study and proposed the pressure drop prediction model as shown in Equation 2 [29].

$$\frac{\Delta p}{\Delta L} = \frac{U}{k_1} + \frac{\rho}{k_2} U^2$$

Where $\Delta p/\Delta L$ is the average pressure gradient (Pa/m), $\Delta L$ is the length of reactor (m), $U$ is the mean flow velocity through the reactor (m/s), $\rho$ is the fluid density (kg/m$^3$), $\mu$ is the fluid dynamic viscosity (N·s/m$^2$), the coefficient $1/k_1$ is viscous permeability (1/m$^2$), and $1/k_2$ is inertial permeability (1/m). In this research, the fluid density and fluid dynamic viscosity is 1.225 kg/m$^3$ and 17.9 $\times$ 10$^{-6}$ N·s/m$^2$, respectively.

For the porous bed composed of uniformly spaced structure, Ergun [30] and Edouard [31] had proposed the Darcy-Forcheimer formula, as shown below:
\[
\frac{1}{k_1} = 2.42 \frac{a_c^2}{\epsilon^3}
\]
\[
\frac{1}{k_2} = 0.36 \frac{a_c}{\epsilon^3}
\]
\[
a_c = \frac{S_s}{V_{Pd/ACFs}}
\]
\[
\epsilon = \frac{V_{total} - V_{Pd/ACFs}}{V_{total}}
\]

Where \(a_c\) is the surface to volume ratio of Pd/ACFs (1/m), \(\epsilon\) is the external porosity (the internal porosity is negligible), \(S_s\) is the external surface area of Pd/ACFs (m\(^2\)/g), \(V_{Pd/ACFs}\) is the external volume of Pd/ACFs (m\(^3\)/g), \(V_{total}\) is the total volume of the ozone removal reactor (m\(^3\)).

In this research, \(S_s = 0.15\) m\(^2\)/g and \(V_{Pd/ACFs} = 20 \times 10^{-6}\) m\(^3\)/g, so the \(a_c = 7500\) m\(^{-1}\). The external porosity of 2 to 4 flat-type ozone removal unit is 0.69, 0.62 and 0.57, respectively. Therefore, the relationship between pressure drop gradient and Equations 6 to 8 can predict flow velocity for 2 to 4 flat-type ozone removal unit.

2 flat-type: \(\frac{\Delta P}{\Delta L} = 7.417U + 10.07U^2\)  
3 flat-type: \(\frac{\Delta P}{\Delta L} = 10.224U + 13.878U^2\)  
4 flat-type: \(\frac{\Delta P}{\Delta L} = 13.157U + 17.859U^2\)

Figure 8 shows the comparison of pressure drop gradient between the experimental data and the predicting value of Equations 6 to 8, the maximum relative deviation is about 7.4% that mainly comes from the ozone removal unit of 2 layers catalyst. While for the 3 and 4 layers catalyst film, the predicting value of pressure drop fits well with the experimental data. It can be seen that the pressure drop of ozone removal unit in this research can be predicted by Darcy-Forcheimer formula.
3.2. Ozone Converter Development and Performance Testing

Figure 9 shows the typical modern aircraft ventilation system in the aircraft, the bleeding air is provided by the engine compressor, ozone removed by ozone converter, cooled by air-conditioning packs located under the wine center section, at last, the clean air was transformed to the cabin. Ozone converter is commonly installed in the aircraft underbody at the duct leading from jet engine compressor to the passenger cabin, it must have excellent ozone removal capability to meet the ozone concentration standards and low resistance to minimize the energy consumption [32, 33].

![Figure 9. The typical ventilation system in the modern aircraft](image)

A flat-type ozone reactor was designed and manufactured based on the above results, and it was housed in a cuboid converter with the dimension of $0.3 \times 0.3 \times 0.3$ m$^3$. Synthesizes the ozone conversion rate and pressure drop in above research, the catalyst film space of 1.5 mm was chosen for the design of ozone reactor.

3.2.1. Ozone Converter Design

Heck and his co-worker [34] proposed the design requirement for ozone converter in the wide body commercial aircraft after a detailed analysis of the in-flight performance. Chen et al. [35] simulate the ventilation of an aircraft cabin mockup with a real MD-82 commercial airliner, mentioned that the minimum air supply is 7.1 L/(per-s) that is equal to 1232 kg/h at the condition of 20°C and 1atm in the aircraft cabin. This value can meet the ASHRAE standard [36]. Therefore, the designed size and performance evaluation index of the ozone converter is based on the parameters in Table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
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<tr>
<td>The minimum air flow (kg/h)</td>
<td>1232</td>
</tr>
<tr>
<td>Temperature (°C)</td>
<td>120-200</td>
</tr>
<tr>
<td>Pressure (atm)</td>
<td>1.6-4.0</td>
</tr>
<tr>
<td>Allowable pressure drop (kPa)</td>
<td>3.4-10.3</td>
</tr>
<tr>
<td>Required conversion (%)</td>
<td>83-93</td>
</tr>
<tr>
<td>Housing diameter (m)</td>
<td>0.2-0.28</td>
</tr>
<tr>
<td>Housing length (m)</td>
<td>0.43-0.56</td>
</tr>
<tr>
<td>Maximum weight (kg)</td>
<td>9-16</td>
</tr>
</tbody>
</table>

A flat-type ozone reactor was designed, which was composed of an external framework and 17 flat-type catalyst units, the schematic diagram and photograph of ozone reactor are shown in Figures 10 and 11, respectively.

The external framework was welded by 12 stainless steel square tubes with the dimension of $0.3 \times 0.3 \times 0.3$ m$^3$. 17 channels were reserved on the vertical square tubes for fixing the flat-type catalyst units. Each catalyst unit was made of a supporting plate and two layers of Pd/ACFs with the dimension of $290 \times 290 \times 8$ (L×W×H) mm, and the catalyst Pd/ACFs were fixed on both side of the plate. 17 flat-type catalyst units were inserted into the channel on the framework to form the reactor, as shown in Figure 10. The space between the catalyst units is 1.5 mm.
The reactor was housed with an entrance and outlet region to form an ozone converter, as shown in Figure 12. In general, the shape of air duct for the fresh bleeding air to the aircraft cabin is circular, so the ozone reactor is connected to the entrance and outlet region by the transition zone from column to square. The diameter and length of the entrance and outlet transition zone is 114 mm and 100 mm, respectively, and the total housing length is 500 mm.
3.2.2. The Performance Test System of Ozone Converter

Figure 13 shows the performance test system for the ozone converter, which was comprised of by-pass Route I and primary test Route II. The parameters of mass flow rate, reaction temperature and initial ozone concentration can be adjusted to simulate the fresh air condition of aircraft cabin.

The stable airflow comes from a high-pressure air source, which is equipped with an electrical heater to control the temperature of airstream. Because the outlet pressure of ozone generator is near to ambient pressure, ozone cannot be introduced into the high-pressure test section directly, so an ejector was equipped to introduce ozone easily.

![Diagram of the performance test system for the ozone converter](image)


Figure 13. The test system for the performance of ozone converter

The bleeding air that coming from the high pressure air source (1) was adjusted by a mass flowrate controller (2), and then which was heated to a given temperature by an electrical heater. An ozone generator was connected to the ozone entrance (4) that is located at the middle of ejector (3) to introduce the ozone into the system. Thus, a fixed flowrate of airstream with constant temperature and initial ozone concentration can flow into the ozone converter (5).

Firstly, the ozone was introduced into the by-pass route I to steady the experimental parameters, and then it was introduced into the route II to test the performance of the ozone converter. The detailed information can be found in the literature [19]. The inlet and outlet ozone concentration were measured by an ozone analyzer (American 2B-technology 106-L with the precision: 0.1 ppb), the corresponding pressure and temperature were measured by pressure transducer (with the accuracy grade of 0.25%) and temperature gauges (with the precision: ±0.5°C), respectively. All the testing data were recorded by a data acquisition instrument every 5 minutes.

3.2.3. The Data Processing

(1) The flow velocity in the ozone converter

The flow velocity $v$ through the ozone converter at each temperature $t$ and pressure $p$ can be obtained with Equation 9.

$$v = \frac{G}{3600 \times \rho A}$$

(9)

Where $G$ is the mass flow rate of the airstream (kg/h), $\rho$ is the density of airstream through the ozone converter at the fixed temperature and pressure (kg/m$^3$), $A$ is the flow cross-sectional area in the designed ozone converter (m), in this research, $A=0.03$ m$^2$. The density $\rho$ can be obtained by Equation 10.

$$\rho = \rho_0 \left(\frac{273}{273+t}\right) \times \frac{p}{0.1013}$$

(10)

Where $\rho_0=1.29$ kg/m$^3$. 
(2) The pressure drop of ozone converter

The pressure drop (Δp) of the ozone converter was obtained by the inlet and outlet pressure difference (p1-p2), where p1 and p2 are the inlet and outlet static pressure of ozone converter (kPa), respectively.

(3) The minimum flow velocity in the ozone removal reactor

The minimum flow velocity was gotten by the minimum air supply of 7.1 L/(per-s) that is equal to 1232 kg/h at the condition of 20°C and 1 atm, according to the Equation 10, ρ20°C=1.23 kg/m³. The minimum flow velocity flow through the ozone reactor is 2.30 m/s.

3.2.4. The Performance Test of Ozone Converter

The ozone converter performance was tested under the mass flow rate of 306-680 kg/h at the temperature of 20°C, 90°C and 150°C. The corresponding velocity range is 2.30-5.12, 2.91-6.49 and 3.14-7.00 m/s, respectively. So the flow velocity in experiment meets the flight requirements. For the commonly commercial airliner, the ozone concentration outside the aircraft cabin is in the range of 0.5-1 ppm at the cruising altitude [37], so the initial ozone concentration was 1 ppm in this study.

(1) Ozone removal performance

Figure 14 shows the variation of the outlet ozone concentration with time over ACFs or Pd/ACFs at different flow velocity and temperature. At the time of 0 min, ozone with different initial concentration was introduced into the system and labeled with a dash line. Figure 14(a) and (b) focus on the variation of ozone concentration at room temperature by using the ozone converter with ACFs and Pd/ACFs, respectively. Ozone concentration dropped quickly as soon as the airstream flow through the ozone converter. The ozone concentration using ACFs decreased from 1.0 to 0.085 ppm at the flow velocity of 2.30 m/s, and it increased to 0.12 ppm as the flow velocity rose to 5.12 m/s. While for the ozone converter with Pd/ACFs, the ozone concentration dropped to 0.13 ppm after 40 min under the same experimental conditions, then it increased to 0.14 ppm when the flow velocity increase to 5.12 m/s. Due to the nanoparticle catalyst palladium occupied the adsorption site of ACFs, the performance of ozone removal using Pd/ACFs seems to be weaker relative to the ACFs at the room temperature. Also, the effect of flow velocity on ozone removal is minor.

![Variation of the ozone concentration with time at different flow velocity and temperature](image)

Figure 14. Variation of the ozone concentration with time at different flow velocity and temperature
However, the performance of ozone removal increased obviously over Pd/ACFs with the increase in temperature, as shown in Figure 14(c) and (d). The outlet ozone concentration was near to 0 ppm at the temperature of 90 and 150°C, then kept at this value in the rest process of experiment. The effect of flow velocity on the ozone removal is not obvious at the higher temperature. The reason is that the catalyst activity is excited and the catalytic reaction between ozone and Pd/ACFs is promoted by the higher temperature. Therefore, the control step for ozone removal at the higher temperature is mass transfer comparing with that in the lower temperature. The greater the flow velocity is, the more ozone molecules will transfer to the surface of Pd/ACFs and be decomposed.

Figure 15 shows the variation of the ozone conversion rate over ACFs and Pd/ACFs with flow velocity at different temperature. At each experimental condition, the corresponding mass flow rate is constant. The ozone conversion rate over ACFs and Pd/ACFs decreased with the increase in flow velocity at 20°C. It decreased from 93% to 90% for ACFs and from 88% to 86% for Pd/ACFs, respectively, when flow velocity increased from 2.30 to 5.12m/s. The ACFs shows a better ozone removal performance than that of Pd/ACFs at 20°C. the reason is that ozone removal mainly come from the adsorption of ACFs at room temperature. The nanoparticles palladium occupied the micro pore of ACFs and caused the decrease of adsorption surface area, which lead to the decreasing performance of ozone removal over Pd/ACFs. Katya Milenvoa et al. [38] studied the ozone removal by loading nanometer Cu and TiO$_2$ on activated carbon and found that the specific surface areas of activated carbon decreased, which perhaps cause a poor ozone removal. The results were similar to our present study.

After raise the temperature, the performance of ozone conversion is significantly improved. However, the ozone conversion rate in 150°C is lower than it at 90°C. The reason is that catalytic reaction is a surface reaction, ozone should contact with the catalyst, and then can be decomposed. For the Pd/ACFs, ozone must diffuse through the outer surface of ACF, then pass through the porous structure and interact with the palladium to complete the removal process. Though, the catalyst activity was promoted when the temperature is increased from 90°C to 150°C, the properties of ozone desorption from ACFs was also enhanced. The promoted properties of ozone desorption has a more effect than the increasing catalytic activity on ozone removal at this condition. On the other hands, the flow velocity on the surface of catalyst at 150°C is higher than that at 90°C under the same mass flow rate, which leads to the less contact time of ozone over the catalyst Pd/ACFs. So the ozone conversion rate was lower at 150°C than that at 90°C. Comparing the data in Table 1, the flow velocity and ozone removal rate can meet design requirements.

![Figure 15. Variation of the ozone conversion over ACFs and Pd/ACFs with flow velocity at different temperature](image)

The testing mass flow rate is from 306 to 680kg/h

(1) The pressure drop

The variation of experimental pressure drop in the ozone converter with different flow velocity was shown in the
Figure 16. The corresponding mass flow rate was in the range of 311-1033 kg/h. It can be seen from the figure that the pressure drop increased with the increase in flow velocity, for example, the pressure drop promoted from 0.44 kPa to 4.85 kPa as the flow velocity increased from 2.40 to 7.97 m/s at temperature of 20°C. The maximum pressure drop of the ozone converter reached 6.44 kPa (at 90°C) and 7.51 kPa (at 150°C), respectively. The pressure drop of the designed ozone converter also meets the design requirements in Table 1.

![Graph showing pressure drop vs flow velocity](image)

**Figure 16. Variation of the pressure drop of the ozone converter with flow velocity**

### 3.3. The Limitation of This Study and the Outlook

Although the designed ozone converter can meet the design property index of pressure drop and ozone conversion rate, the weight is 16.8 kg that is 0.8 kg heavier than the maximum weight in Table 1. The construction materials of ozone converter can be substituted by the aluminum or other light materials in the future designed. On the other hands, the effect of temperature and flow velocity on the ozone removal performance was analyzed through experiment, due to the rigorous testing conditions, it is necessary to find the mathematical model and use the technique of computational fluid dynamics (CFD) to research the procession of ozone removal and optimal design of ozone converter in the future.

### 4. Conclusion

In this paper, three-arrangement type of flat catalyst film are used to experimentally explore the ozone removal performance, the influence of flow velocity and temperature on ozone removal capacity and flow resistance characteristic is studied in detail. Based on the above experimental results, the ozone removal unit with 1.5cm space of Pd/ACFs that has an excellent performance to guides the design of ozone converter. At last, the prototype is tested through the testing system, which can simulate the actual work condition of aircraft cabin. From the present study, the following conclusion can be obtained:

- The one-trough ozone removal performance by catalyst Pd/ACFs films shows that the ozone removal unit with the space of 1.5 cm has an excellent ozone removal performance and low-pressure drop to meet the design requirement.
- The Darcy-Forchheimer model of the porous media can predict the pressure drop of the flat-type reactor in this study.
- The designed flat-type ozone converter has the ozone conversion of 97% and the maximum pressure drop of 7.51 kPa at the temperature of 150°C and flow velocity of 10.63 m/s, which can meet the ozone converter design requirements: The ozone conversion rate of 83-93% and the pressure drop of 3.4-10.3 kPa
5. Funding

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6. Acknowledgment

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Integrated Project Delivery Implementation Challenges in the Construction Industry

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Abstract

Huge financial resources are spent in the construction industry all over the world, which are frequently wasted largely due to a lack of proper planning. In recent decades, in an attempt to overcome challenges, various contractual and administrative systems have been used by construction owners/clients. One such system has been Integrated Project Delivery (IPD). Its implementation has, however, experienced drawbacks. Identifying such drawbacks is an initial step in attempting to resolve them, and this paper aims to identify and prioritize the IPD implementation drawbacks in the context of the Iranian construction industry. A comprehensive list of IPD implementation drawbacks is prepared using a questionnaire survey. An in-depth literature review of the IPD concept has been combined with a review of various case studies applying the IPD system. The results were analyzed using the Robust Exploratory Factor Analysis (EFA) method. 22 drawbacks in the Construction Industry were categorized under four themes; contractual, environmental, managerial, and technical. Results show that contractual drawbacks are the most significant. The implication of this research is that identifying and classifying IPD implementation drawbacks provides a useful reference to managers and owners of the construction industry, for identifying and codifying solutions to overcome them.

Keywords: Integrated Project Delivery (IPD); Challenges; Project Key Stakeholders; Construction; Robust Exploratory Factor Analysis (EFA).

1. Introduction

Demand for construction has been high, however, due to unsophisticated communications among its practitioners, it has been found to have very low efficiency [1]. The industry’s owners should have a common language, to resolve management and communication problems, and to reduce inefficiency and confusion [2]. The construction industry has a great impact on the global economy and other industries; however, in the product delivery sector, information technology, and design, it suffers some challenges [3]. Given the huge volume of this industry, its changes have been so limited; so it has a low productivity. In this industry, billions of dollars are spent for project delays, duplication of work, changes, loss of materials, etc. [4]. In the United States, almost $1 billion is spent on construction per year, and about 30% of this amount is wasted [5]. What is so significant in this regard, is financial success of the construction projects; however, the self-centered behavior of stakeholders under the traditional systems results in not achieving the desired outcomes [6, 7]. On the other hand, funds of large projects are limited; therefore, the industry’s owners attempt to attract

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private sector investment. Consequently, maintaining this capital and careful planning for it is significant [8]. Project implementation method is a major factor in project success [9]. Project implementation is a process, in which a project is defined, designed, constructed, and maintained [10].

Integrated Project Delivery (IPD) is a project implementation approach, which creates directions for change to improve contracts, to make planning more precise, to make cooperation more productive, to make communications more effective, and generally to eliminate lag of the construction industry relative to other sectors [2]. This approach uses a participatory method; therefore project objectives are realized more effectively [11]. The Architectural Institute of America (AIA) defines IPD as: "it is a project implementation system that benefits from talents and constructive cooperation and experiences of all project stakeholders, through using a multilateral contract in an integrated way, to achieve success [12]. In fact, IPD is a philosophy, an intellectual attitude. Stakeholders share their information and resources to meet their obligations [13].

The IPD approach includes the following principles: mutual trust and respect, which are based on teamwork and team building. Common profit and loss, according to the terms of presence of each of stakeholders in the project, their profit and loss will be based on an agreement. Free communications are defined properly as a result of responsibilities, and conflicts among the members are minimized. Early introduction of target leads to strengthening incentives. Leadership is assigned to a qualified person. Strengthening planning, this results in correct timing and better control of costs. Creativity in decision-making, which is due to free exchange of views. Early involvement in the project, which leads to a better sharing of experiences. Appropriate technology, which is considered as one of the significant factors of IPD implementation [12]. Applying these principles leads to removing borders existed in the traditional contracts. Therefore, objectives and responsibilities are defined more clearly [14]. Now, IPD approach is not used widely [15]. However, according to the studies conducted by El Asmar et.al. In the field of comparing traditional methods and IPD, it is hoped using IPD in future is growing, because increasing product quality follows proper environmental and financial performance, reducing project changes, etc. [6].

Identifying and reviewing challenges and introducing solutions to resolve them, will be a useful step toward IPD implementation. In the research conducted by Kahvandí et al. [16], some IPD challenges were identified as macro factors, such as capital factors, organizational factors, and environmental factors, through examining researches conducted during 2001 and 2016. These factors include several challenges. Finally, 44 items obtained through meta-synthesis approach, which is used in this study.

The necessity of research is important considering the two factors "time" and "place". Time was important in this research because the developments in the construction industry are happening quickly and costs are rising. The place is also important because developing countries are looking for solutions to the advancement of the construction industry. This paper aims to evaluate IPD implementation challenges in the construction industry in the mass-housing projects. Using IPD challenges evaluation and classification, more significant challenges can be identified. Comprehensive and accurate planning can be effective in coping with these challenges, and it will help enhancing knowledge integration [6, 17]. Consequently, it leads to massive savings in products and improving their quality, which results in enhancing life quality. Moreover, it will lead to increased energy management capabilities, increased safety, increased productivity, and sustained environment [18, 19]. The next section examines research background.

1. Literature Review

In this section, the definitions related to IPD are reviewed, also the importance of considering IPD implementation challenges are illustrated.

1.1. Integrated Project Delivery (IPD)

During recent years, the Construction Industry has seen significant progress in implementing project implementation systems. These advances have been the implementation of the IPD approach which may help to solve many age-old problems. The necessity to increase productivity in the Construction Industry, stakeholders’ demands, market needs, increasing complexity of technology in the Construction Industry, and the need for buildings' stability, are some of the features that justify the necessity of applying IPD [2, 20]. In the process of implementing a project, employer, consultant, main contractor, sub-contractors, suppliers, and manufacturers play significant roles together [9]. Various organizations are also considered as key stakeholders of a project. An important step in continuing the work is establishing effective relations among these stakeholders. In the traditional systems, due to the restrictions imposed by contracts, these relations have various limitations, particularly for the construction contractors with designers, and with maintenance contractors, which leads to developing conflicts [3]. With the help of IPD that creates trust and clarity in relations, and makes the stakeholders participate in the initial phases of the project, these conflicts may be reduced [21].

The IPD approach has several advantages, some of which are defined by Collins and Parrish 2014 and the Architectural Institute of America (AIA 2007) [22], and some others are classified in the implementation section in the
reviewed case studies [21]. Some of these advantages are increased accuracy of project control, increased product quality, reduced construction time, better leadership, etc., which are the most significant factors for project employers/clients [23]. Reducing costs, accurate planning, participation in profits, etc. are of considerable importance for contractors [8]. Items such as reducing claims, reducing waste of resources, reducing wastes and others are also common between the contractor and the employer [24]. However, what is important for designers is designing in line with the needs of the employer and improving design quality [4]. Due to the presence of the contractor in the initial stages of the project, reducing change orders and reducing the request for information are in favor by the contractor [11]. [12]. Finally, advantages such as public decision-making, developing long-term relations among stakeholders, the possibility of using integrated software, and others are also common among the employer, contractor, and designer [25].

The Architectural Institute of America (AIA) has developed various contract forms to implement IPD [26]. What is so significant in these multilateral standard contracts, is the high rate of agreement among the stakeholders [27]. Typically, in IPD, there is a contract for the entire project, and it includes all of the project stakeholders. In this regard, cooperation and coordination for the entire project are improved significantly [28].

1.2. IPD Implementation Challenges

What is significant is that evaluating IPD advantages alone is not sufficient for its effective implementation [29]. Researchers have evaluated various aspects of its implementation. Numerous studies have examined IPD implementation challenges considering the conditions of different regions of the world and their governing rules on the Construction Industry. Kent et al., through examining the attitude of experienced experts in the Construction Industry, have concluded that they are optimistic about IPD implementation and its consequences. However, advanced information and technological applications are prerequisites for IPD implementation [19, 30]; so supplying it, requires many possibilities. On the other hand, cultural challenges and organizational resistance to changing their previous trends are among other factors mentioned by Kent et al. (2010) [31, 32]. Resolving these challenges by stakeholders provides significant results of IPD implementation. For example, in a medical center project in San Francisco, implementing IPD saved about $1 million in the electrical equipment sector and about $5 million in the mechanical sector. In the medical center building in Fairfield, the initial budget was estimated at $12 million, which was reduced to $19 million by applying IPD. In both projects, by agreement of the employer, the contractors of the maintenance section were presented in the initial stages of the project as a consultant [25]. Another example is Cathedral Hill Hospital in the United States. In this project, there was the problem of determining final costs, which was completed at the right cost, by using IPD contract and entrance of contractors with the responsibility of supplying resources and equipment [21, 33].

During studies performed Ghassemi and Greber (2011), IPD implementation challenges were divided into four main categories, including cultural; these challenges refer to the reluctance of industry’s owners to change the traditional methods [26, 34, 35]. Changing their attitude needs a hierarchical because the construction companies have accustomed to their limited leadership [27]. Financial challenges; selecting compensation for damages is a big challenge in IPD implementation [21, 26]. Legal challenges; issues related to responsibility and insurance are among other challenges of IPD implementation. Current insurances don’t fully support delegating responsibility [21, 26, 36]. Technology, cooperation, and integrated use of technology in IPD implementation are considered as significant steps in its implementation. Training at organizational levels is also very important in resolving cultural challenges [31, 37].

Manning in the study conducted in 2012 concluded that the necessity of IPD implementation is applying advanced technology in the architectural and engineering aspects of construction [8, 23]. Employer liability insurance with special conditions is also significant because if insurance companies don't support IPD, sharing profit and loss will not be implementable. Another challenge in this regard is lack of training and introducing IPD to employees [8, 12]. According to the conducted studies by Nejati et al. (2014), IPD implementation challenges in the mass housing projects include: distrust of stakeholders to each other, the right of final decision-making about particular issues just for the employer, high levels of discretion of the employer, lack of familiarity with BIM, unwillingness to use new methods in implementing contracts, lack of financial transparency, unwillingness of the employer to share project profit with the consultant, lack of special plan for profit and loss, lack of transparency in the costs of contractors, unwillingness of the contractor to participate in the design stage, lack of knowledgeable people to resolve claims and lack of sufficient knowledge about developing an industrial method among consultants [14, 38, 39].

Evaluating challenges was also conducted by Kahyandi et al. (2019) IPD challenges included macro factors such as capital, organizational, and environmental factors. Capital factors include financial challenges; and organizational factors include managerial, contractual, educational, communication, and technology challenges. Environmental factors included cultural, legal, and political challenges. Finally, 44 codes were obtained using meta-synthesis approach [16].

In this study, these 44 codes were analyzed. The significance of this study is for rooting IPD challenges, and to highlight them for decision-makers to be able to resolve them more effectively. Resolving challenges will pave the road for better IPD implementation [16, 40]. The next section introduces the research method.
2. Research Methodology

The research steps are shown in Figure 1. In Appendix I, a 44-item questionnaire is presented based on the literature review and investigated IPD projects’ challenges. The survey instrument asked the respondents to rate the importance of every 44 challenges using a nine-point scale with items ranged from 1 (strongly low) to 9 (strongly high). In this research, activists in the field of construction in Iran with experience using IPD and project managers and employers and consultants and contractors with a high academic level and more than five years of experience participated.

To gather data from the respondents, first, the companies and related experts were identified. They were asked to fill the questionnaire and finally, they completed the questionnaires (Appendix I). In total, 500 questionnaires were sent out to the respondents, 245 questionnaires were gathered, and 225 usable questionnaires were included in the data analysis (response rate: 0.45). The sample size of 225 seems to be adequate for conducting robust EFA (recommended ratio of 5:1) [41].

EFA is used in this study which is a frequently used method to discover patterns of multidimensional constructs that are subsequently used for the development of measurement scales. Its principal objective is to reduce the number of observed variables to fewer factors to enhance interpretability and detect hidden structures in the data. In other words, EFA’s purpose is to ascertain the most parsimonious number of interpretable factors required to explain the correlations among the observed variables, with or without underlying theoretical processes in mind. Thus, EFA is a method for identifying the factor structure of a set of multiple indicators or variables without imposing an a priori structure on the factors. EFA is performed at early stages of research to consolidate variables and generate new hypotheses about underlying theoretical processes [42]. Here, robust EFA [43] was employed to perform the analysis which is less influenced by data outliers and data heterogeneities.

3. Results

Before conducting robust EFA analysis, a test was conducted to verify the adequacy of the data. The Kaiser-Meyer-Olkin (KMO) was calculated to ensure sampling adequacy. The KMO for the sample is 0.75 which is above the "Mediocre" threshold of 0.5 [43]. Furthermore, the authors performed a Bartlett sphericity test, which was statistically significant (p < 0.05), indicating the eligibility of the data. Then, we used a Shapiro–Wilk test to determine whether the sample had a normal distribution. It was found that none of the variables were normally distributed. Thus, principal component analysis (PCA) was the choice for the factor extraction method as proposed in robust EFA. The rotation method should also be selected for the robust EFA purpose. Oblimin rotation, which is suggested in robust EFA was used in this research [43]. Finally, the number of factors to be extracted from the data were determined based on Eigen values greater than one, and an absolute factor loading values greater than 0.6 [44]. As a result, two out of 22 factors were dropped from the initial pool and remained 22 factors were grouped into four components. The results can be seen from Table 1.

<table>
<thead>
<tr>
<th>Items</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Lack of mutual trust among project key stakeholders regarding managerial and financial issues</td>
<td>0.90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Lack of appropriate policies and current construction contractual strategies</td>
<td>0.83</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Lack of identical contracts among subcontractors, such as IPD approach</td>
<td>0.72</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Tendency to use conventional contractual methods and resistance to new ideas</td>
<td>0.81</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. Lack of proper definition of responsibilities of each of parties of the contract</td>
<td>0.89</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6. Lack of motivation for investors to use modern contracts, such as IPD approach</td>
<td></td>
<td>0.74</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. Lack of control and strong management of the employer</td>
<td></td>
<td></td>
<td>0.81</td>
<td></td>
</tr>
</tbody>
</table>
8. Lack of proper orientation for future and not paying attention to future development, particularly in the governmental projects 0.75
9. Lack of familiarity of contractors with IPD approach 0.83
10. Lack of conditions for the insurance to cover the entire project in the country, according to new contractual systems 0.82
11. Lack of conditions for the insurance to cover the responsibilities according to new contractual systems for the contractor 0.84
12. Non-participation of governmental agencies in construction, according to the governing rules in the governmental contracts 0.86
13. The challenge of selecting compensator for financial losses 0.75
14. Inconsistency in project management 0.83
15. Poor matrix structure in project-based organizations 0.84
16. Lack of sufficient knowledge of investors about new successful contractual systems all over the world 0.81
17. Lack of holding training courses for investors about defining and stating the advantages of new successful contractual systems all over the world 0.78
18. Poor information sharing among different phases of the project 0.80
19. Lack of proper definition of teamwork culture among project key stakeholders 0.74
20. Lack of integrated collaboration among key stakeholders, due to lack of the necessary technology 0.75
21. Lack of using BIM as an appropriate instrument to implement IPD approach 0.80
22. Lack of sufficient knowledge about design and construction and maintenance among employer agents 0.87

| % of variance | 23.94 | 20.04 | 16.12 | 12.45 |
| Cumulative % | 43.98 | 60.1 | 72.55 |

To indicate the meaning of the components, they have been given short labels indicating their content. Since the results of this stage were open to several interpretations, the authors decided to use experts’ opinions. Consequently three IPD project managers were selected, who had the experience of using IPD with high academic level and more than five years of experience. Based on the discussions on the factors’ meanings in each component, four “Managerial”, “Environmental”, “Contractual”, and finally “Technical” labels were assigned to the extracted components. The final results are shown in Table 2.

**Table 2. Extracted components and their related factors**

<table>
<thead>
<tr>
<th>Components’ Title</th>
<th>Factors’ title</th>
</tr>
</thead>
<tbody>
<tr>
<td>Managerial</td>
<td>The challenge of selecting compensator for financial losses</td>
</tr>
<tr>
<td></td>
<td>Inconsistency in project management</td>
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<td></td>
<td>Poor matrix structure in project-based organizations</td>
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<td>Lack of sufficient knowledge of investors about new successful contractual systems all over the world</td>
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<td>Lack of holding training courses for investors about defining and stating the advantages of new successful contractual systems all over the world</td>
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<td></td>
<td>Poor information sharing among different phases of the project</td>
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<td></td>
<td>Lack of proper definition of teamwork culture among project key stakeholders</td>
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<tr>
<td>Environmental</td>
<td>Lack of motivation for investors to use modern contracts, such as IPD approach</td>
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<td></td>
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<td></td>
<td>Non-participation of governmental agencies in construction, according to the governing rules in the governmental contracts</td>
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<tr>
<td>Contractual</td>
<td>Lack of mutual trust among project key stakeholders regarding managerial and financial issues</td>
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<td>Tendency to use conventional contractual methods and resistance to new ideas</td>
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<td></td>
<td>Lack of proper definition of responsibilities of each of parties of the contract</td>
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</tbody>
</table>
4. Discussion

Information classification has advantages even at the level of global investment. It also influences the level of accessibility to minimize the effects of risks. The classification of data in each section is part of the needs of each system. In previous studies, there were categories for challenges to IPD implementation but in this research, the authors have tried to make the categorization more fully with the new methodology. According to the percent of variance mentioned in Table 1, the order of factors is: 1) contractual, 2) environmental, 3) managerial, and finally 4) technical.

According to what is achieved in this classification, contractual challenges with 23.94% of explained variance are the most significant challenges. This is due to the fact that the IPD system at first includes a contractual system that includes the project lifecycle [12]. Consequently, contractual challenges cause problems in IPD implementation due to various factors. Mutual trust in contracts, particularly in the IPD system, is one of the conditions for its success. According to the behavioral and communicative principles in IPD, i.e., the culture of mutual respect and cooperation and participation and transparent relations and open communication, etc. trust and IPD should cooperate with each other [45]. In a medical project in Denver, with 430,000 SF area and $160 million costs, and 24 months timing, the IPD approach was performed successfully and reduced costs by 26% relative to similar contracts such as Design-Bid-Build (DDB) with 13% cost reduction, and Design-Build (DB) and Construction Management at Risk (CMR) with 17% cost reduction [32]. In another project in Phoenix, United States, Walter Cronkite School of Journalism, there was state law prohibits for IPD implementation. Therefore, design and construction contract was used with respecting behavioral principles and some IPD contractual principles. Finally, the project was completed without wasting money [25].

In each project, two sets of factors are effective; one set of internal and intra-organizational factors, which here are referred to as managerial factors; and another set of external and environmental factors. Environmental factors (the second most important category with 20.04% of variance) are largely influenced by state laws and cultural factors [14]. Reforming state laws to implement IPD, is very significant. In a project in the United States, project contract was signed using design and construction, due to State challenges. In this contract, two main factors of IPD were used; i.e., the participation of key stakeholders from the beginning of the project and their joint decision-making and supervision [26]. Issues related to insurance and rules related to compensating losses are significant in projects.

In IPD, conflicts will be minimized, by reducing some of the authorities and increasing cooperation. Current insurance contracts have some terms that make responsibilities more complicated; consequently, stakeholders' authorities for IPD implementation are not specified explicitly [46]. Therefore, in recent years, no IPD particular insurance contract has been developed, IPD users have to change the way of using insurance. For example, the Autodesk One Market project in the United States was performed in a building with 40,000 SF areas and about $10 million budget and 9.5 months planning. The project team used insurance contracts, with this difference that all of the claims were deleted by the agreement of all stakeholders, except items related to fraud and neglect in the job [26]. The Proper definition of IPD and introducing its advantages will be effective to attract the attention of insurance companies and banks to compensate financial losses. In some projects, resolving the challenge of the unfamiliarity of contractors with IPD approach has had significant consequences. For example, in the Cardinal Glennon Children’s Hospital project in the United States, holding training courses convinced stakeholders to use an integrated contract to overcome the problem of lack of flexibility of traditional contracts and reducing project’s complexities [25].

The results of this study show that managerial category is the third most important set of challenges. Lack of coordination in selecting the compensator of financial losses is one of the challenges that violates IPD principles, because in IPD, a high level of coordination and cooperation in all units is required, and profit and loss are divided jointly among the stakeholders [47]. On the other hand, a part of the managerial challenges is related to intra-organizational issues, which are solvable by basic changes in the structure of project-based organizations and creating coordination and proper determination of duties. The training sector is one of the most important sectors that started to use new systems. Developing this sector can be helped with real examples of successful IPD implementation in other projects [12]. These challenges are rooted in the traditional systems of project implementation; because in such systems, relations are so limited and after the completion, each project’s phase is given to other stakeholders. Therefore, cooperation and coordination among stakeholders were very low [26]. On the other hand, it can be stated that resolving managerial challenges, due to their significance, provides a clear vision for solving other challenges.

Using technology, in any industry can be challenging. In IPD, information integration needs to resolve the challenges [11]. Building Information Modeling (BIM) is an instrument required for information integration capability in IPD. However, lack of sufficient knowledge and lack of applying it in the country has been shown to make IPD implementation challenging. In similar projects using IPD, such as Autodesk One Market in the United States,
cooperation between BIM and MEP made the maintenance sub-contractors to enter the design stage and save the costs and time significantly [26]. In fact, BIM creates a platform to share information and is very effective in the field of responsibilities and ownership of activities [48]. In some projects, time and costs were saved significantly [26].

The classification obtained in this study is largely in line with the results achieved from the previous studies [14], [26]. Because of these macro factors, besides their various impacts on the construction industry, are the concerns of many owners of this industry and is common among all of them. Consequently, classifying macro factors is significant, because it defines a useful database, which can be effective for planning and resolving the challenges and reducing negative effects [46, 49]. On the other hand, it can be stated that in this study a summary of significant items has been determined and updated, to be available. The classified macro factors include many subsets in the construction industry, all of which are effective factors in this Industry. Considering the large investment in IPD, the smallest changes in each of them, could result in significant savings in time and cost in the construction. In this paper, problems were identified with the study of library studies and the question of project stakeholders. These items were ranked by a comprehensive survey, to determine the most significant items. Identifying these items and evaluating them, provides this possibility for industry’s owners to try to resolve them, to implement IPD in the best way.

5. Conclusion

In this study, a comprehensive list of challenges to apply IPD was developed in the form of a questionnaire, through use of a comprehensive literature review and examination of case studies using IPD. This was followed by a survey of project managers and employers and consultants and contractors active in the field of construction. The obtained results were analyzed using the Exploratory Factor Analysis method. Among 44 questioned items, 22 items of IPD implementation challenges in the construction industry were prioritized. Then, they were classified by some experts of the Industry, into four categories or macro factors including contractual, environmental, managerial, and technical ones. These were analyzed in the previous section. The results showed that contractual macro factors are considered as IPD implementation challenges in the Iranian construction sector. What was significant in this study was that resolving contractual challenges is very effective in resolving environmental, managerial, and technical challenges. What is significant is that IPD successful implementation requires resolving basic challenges and resolving those challenges faced during its implementation. The conditions of projects are unique; so considering its situation, there are different and similar solutions to resolve them. However, what is obtained from the experiences of recent years about IPD implementation is its' significant success is saving time and costs in the project lifecycle. Classifying these challenges can be used by industry’s owners to help resolve them.

The main limitation of the study was the lack of case studies using a multi-party IPD contract. There were also IPD projects that were currently under construction which further reduced the sample size of IPD projects. For future studies, these classified challenges can be evaluated through case studies implemented by IPD and countries such as the United States, Australia, Canada, and others that implement IPD contracts, or various solutions can be presented for resolving them by surveying experts. By examining more case studies, other potential challenges of projects can be analyzed that are developed due to their special conditions. Moreover, future research can move beyond listing challenges and could explore the interrelationships between them.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Appendix I: The questionnaire items

A questionnaire to determine the barriers affecting the implementation of modern systems in construction contracts.

The current questionnaire is designed and evaluated to examine the effective barriers to the implementation of modern systems in construction contracts. Given that you have the experience of participating in the consultation or implementation of the construction project implementation system, please comment on the questions posed in this questionnaire.

What is your gender?
- Male  - Female

How old are you?

What is the highest level of education you have completed?
- High school graduate  - Two-year/associate’s degree or some college  - Four-year or bachelor’s degree  - Graduate or master’s degree  - Professional degree (for example, MD, JD, DDS)  - Postgraduate degree or PhD

Please highlight the importance of the obstacles listed in the following table in implementing the new project implementation systems, such as integrated project delivery, by choosing an option (from 1 (strongly low) to 9 (strongly high)).

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<th>Items</th>
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<td>2. Lack of integrated collaboration among key stakeholders, due to lack of the necessary technology</td>
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<td>3. Non-participation of governmental agencies in construction, according to the governing rules in the governmental contracts</td>
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<td>4. The challenge of selecting compensator for financial losses</td>
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<td>6. Disappointment to choice the suitable construction team</td>
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<td>8. Unclear duty of each of parties</td>
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<td>9. Lack of using BIM as an appropriate instrument to implement IPD approach</td>
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<td>10. Lack of sufficient knowledge about design and construction and maintenance among employer agents</td>
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<td>11. Lack of holding training courses for investors about defining and stating the advantages of new successful contractual systems all over the world</td>
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<td>12. Poor information sharing among different phases of the project</td>
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<td>13. Lack of existence of like IPD contracts</td>
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<td>14. Lack of mutual trust among project key stakeholders regarding managerial and financial issues</td>
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<td>15. Retaining the right of ending decision for the owner</td>
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<td>16. Lack of conditions for the insurance to cover the entire project in the country, according to new contractual systems</td>
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<td>19. Lack of direction in payment systems</td>
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<td>20. Poor matrix structure in project-based organizations</td>
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<td>23. Lack of direction in managing project organization</td>
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<td>Tendency to use conventional contractual methods and resistance to new ideas</td>
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<td>Lack of proper definition of responsibilities of each of parties of the contract</td>
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<td>Lack of existence of preparation materials in country</td>
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<td>Lack of integrated interoperability because of lack of essential technology</td>
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<td>Inconsistency in project management</td>
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<td>Lack of existence of right stakeholders in a place through all phases of the project</td>
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<td>Changes in the original design in the construction phase</td>
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<td>Corporations get used to traditional systems</td>
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<td>Disinclination of stakeholders to contribute in a project with common benefits</td>
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<td>Opposition of stakeholders to take risk</td>
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<td>Lack of appropriate policies and current construction contractual strategies</td>
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<td>Contracts that make several units to follow it</td>
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<td>Lack of control and strong management of the employer</td>
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<td>Lack of existence of appropriate conditions for IPD implementation in the public Construction part</td>
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<td>Lack of proper orientation for future and not paying attention to future development, particularly in the governmental projects</td>
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<td>Lack of familiarity of contractors with IPD approach</td>
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<td>Lack of transparency in costs</td>
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<td>Specific supplies of insurance to the full project</td>
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<td>Lack of motivation for investors to use modern contracts, such as IPD approach</td>
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Non-deterministic Approach for Reliability Evaluation of Steel Portal Frame

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Abstract

In recent years, more researches on structural reliability theory and methods have been carried out. In this study, a portal steel frame is considered. The reliability analysis for the frame is represented by the probability of failure, \( P_f \), and the reliability index, \( \beta \), that can be predicted based on the failure of the girders and columns. The probability of failure can be estimated dependent on the probability density function of two random variables, namely Capacity \( R \), and Demand \( Q \). The Monte Carlo simulation approach has been employed to consider the uncertainty the parameters of \( R \), and \( Q \). Matlab functions have been adopted to generate pseudo-random number for considered parameters. Although the Monte Carlo method is active and is widely used in reliability research, it has a disadvantage which represented by the requirement of large sample sizes to estimate the small probabilities of failure. This is leading to computational cost and time. Therefore, an Approximated Monte Carlo simulation method has been adopted for this issue. In this study, four performances have been considered include the serviceability deflection limit state, ultimate limit state for girder, ultimate limit state for the columns, and elastic stability. As the portal frame is a statically indeterminate structure, therefore bending moments, and axial forces cannot be determined based on static alone. A finite element parametric model has been prepared using Abaqus to deal with this aspect. The statistical analysis for the results samples show that all response data have lognormal distribution except of elastic critical buckling load which has a normal distribution.

Keywords: Reliability Analysis; Monte Carlo Method; Matlab; Abaqus.

1. Introduction

The design of engineering structures is usually associated with a significant level of uncertainties due to limited information in the process of estimating the structural parameters. The impact of uncertainties needs to be quantified and propagated to obtain the reliability of a structural system Morio and Balesdent (2016) [1]. In practice, most engineering design of structures are based on deterministic parameters and often do not consider the variations in the material properties and the geometry of the structure. Ebenuwa and Tee (2019) stated that the determination of structural performance based on the deterministic model is undoubtedly a simplification because physical measurement always shows variability and randomness [2].

In many circumstances, it is impossible to describe the response of structural systems mathematically because of these uncertainties. Even after finding a mathematical model to predict the behavior of the system, there is no closed form solution for solving the equation. In such cases, simulation is one of the most applicable techniques to acquire the required information. Simulation is a special technique to approximate the quantities that are difficult to obtain.
Monte Carlo simulation method is one of the well-known and common procedures in solving complex engineering problems Melchers and Beck (2017) [3].

The origin of Monte Carlo began in the 1940s by three scientists, John von Neumann, Stanislaw Ulam, and Nicholas Metropolis while working on a nuclear weapon project called the Manhattan Project. They conceived of a new mathematical method that would become known as the Monte Carlo method. Stanislaw Ulam coined the name after the Monte Carlo Casinos, located in Monaco south of France. Soon, applications started going up in all sorts of situations in business, engineering, science and finance [4].

Theory and methods for structural reliability have been developed substantially in the last few years and they are actually a useful tool for evaluating rationally the safety of complex structures or structures with unusual designs Gordini, et al. (2018) [5]. Recent evolution allows anticipating that their application will gradually increase, even in the case of common structures Cardoso et al. (2008) [6].

Zhange and Zhou (2013) studied the system reliability analysis of a 3D steel frame designed used AISC LRFD with respect to the collapse limit state under the dead and live loads. They evaluated the system reliability of the frame for two cases, Case 1 was ignoring the spatial variation of the live load and Case 2 was considering it. The results showed that the reliability index for Case 1 is slightly higher than Case 2. This indicated that the spatial inconsistency of the live load decreases the system reliability of the frame [7].

In 2016 Klink and Silva assessed the reliability and security of a steel I-beam profile subject to an applied bending moment. The purpose was to evaluate the suitability of the beam in handling specific project stresses. The Monte Carlo method was used, to obtain the probability of structural failure. Based on the analysis, the I-beam was oversized, thus, can be submitted to increase loading stresses without damaging the global structure [8].

The probability of failure for steel column and beam had been examined by Manjunath and Sagar in 2017. The probability modelling adopted using Matlab and Monte Carlo method to generate pseudo-random numbers for the parameters considered in the statistical analysis. The reliability analysis show that the probability of failure for column was greater than the probability of failure for beam [9].

Zhang et al. (2018), examined the system reliabilities of a number of simple yet representative structures subjected to gravity loads, including a continuous beam, a portal frame that fails by elastic instability, and three related frames with various load redistribution capacities. The research provided an overview of the strengths and system reliabilities of these structures when designed either by the second-order inelastic method or by LRFD in AISC 360-10. Based on the system reliability analysis results for the five structures, some general observations were made. As designed by LRFD, the frames system reliability indices were quite scattered. The system-based design by inelastic analysis is well able to achieve identical system reliabilities than present member-based LRFD. This is to be predictable given that the inelastic method is explicitly based on overall system behaviors. The reason that the inelastic method leads to lower system reliabilities than LRFD was that the inelastic analysis, in contrast to LRFD, leaves little reserve strength after first yielding in the system [10].

In 2019, the system reliability analysis based limit state design criterion for 3D steel frames under wind loads had been studied by Wenyu et al. Through the Monte Carlo technique, the probabilistic characteristics of the ultimate lateral strengths of the frames are determined. It was found that despite the differences in structural configuration, system size and degree of redundancy of the frames, their ultimate lateral strengths share similar probabilistic characteristics, i.e., they can be generally described by lognormal distributions [11].

The behavior of steel frame is generally assessed based on their strength and their elastic deformations In addition to the deterministic aspects that discussed in mechanics of material, the strength and deformation of steel frames have random parts due to the scatter in the dimensions, material properties, and the applied load. These random aspects can be simulated in terms of the probability density functions that either obtained from real experimental data on the member scale level or from the simulation that based on data of sectional level [12].

This paper starts with data gathering from literature for the variation in cross-section dimensions of frame elements, the variation in the elastic modulus and yield stress of the material, and the scatter in the applied loads. Based on these data, it has been found that the variation in the sectional dimensions, elastic modulus, yield stress, and dead loads are normally distributed while the lognormal and extreme type I (Gumbel) can be adopted for the variation in the length and live loads respectively.

Monte Carlo simulation has been used to generate a sample for the parameters that effected on the frame behavior. Two samples have been generated first one is the demand sample while the second one is the capacity samples. These samples had been presented and summarized in the form of histograms. The generated sample has been statistically tested with the $\chi^2$ test. Base on limit state function, these samples have been used to estimate the probability of failure for the portal frame. This study innovatively concerns with the randomness in structural parameters and how these
randomness effects on structure reliability by determining the probability of failure and reliability index using Monte Carlo simulation method and Approximate Monte Carlo simulation method.

2. Uncertainties in Engineering System

Every structure may contain some failed elements which lead to the whole structure failure. The probability of failure for structure can be predicated established on the failure of its elements. Hence, it is significant in reliability analysis to determine the probability of structure elements failure. First and second-order of reliability method and Monte Carlo methods can be used to analyze the reliability of elements [13]. For the statically indeterminate steel portal frame of this paper, the girder and columns failure have been used to estimate the frame probability failure.

The uncertainties included in the building engineering can be categorized according to their source into natural hazards and man-made hazards. Natural hazards may be resulted by wind, seismic, temperature differentials, snow load, or ice accretion. The natural variations of structural properties such as strength, stiffness and loads can be classified within the natural hazards. On the other hand, from a structural point of view, the man-made hazards can be sub classified into two classes: from within the building process and from outside the building process. The second one includes uncertainties due to fires, gas explosions, collisions, and similar causes, while the first one includes uncertainties due to acceptable practice and those caused by departures from acceptable practice [14]. This paper concerns with the natural hazard aspects due to change in stiffness, strength, and applied loads.

3. Performance Functions

The limit state function or performance function represents the relation between capacity, $R$, and demand, $Q$.

In this paper, the serviceability limit state deflection function and ultimate moment limit state function have been studied for the frame. The ultimate limit states can be used to determine the safety margin. The performance function can be written as follows:

$$g(R, Q) = R - Q$$  \hspace{1cm} (1)

The structure is classified safe when $g \geq 0$ while it is unsafe when $g < 0$. Mathematically, the failure probability $P_f$ is equal to the probability of $g < 0$ [2]:

$$P_f = P(g < 0) = P(R - Q < 0)$$  \hspace{1cm} (2)

If $R$ and $Q$ have probability density functions (PDF) indicated in Figure 1, the quantity $R-Q$ would be a random variable also with its own PDF. As shown in Figure 1, the probability of failure would correspond to the shaded area.

![Figure 1. PDFs of load, resistance, and safety margin [15]](image)

A direct determinate of $P_f$ from Equation 2 is relatively difficult. Therefore, it would be more appropriate to express structural safety in the expression of a reliability index $\beta$, which can be described as the shortest distance from the origin to the failure limit. When $R$ and $Q$ are uncorrelated the $\beta$ would be the inverse of the coefficient of variation of the Equation 1 [14] and the reliability index is related to the probability of failure by:

$$\beta = -\varphi^{-1}(P_f) \quad \text{or} \quad P_f = \varphi(-\beta)$$  \hspace{1cm} (3)

From a statistical point of view, the PDF of $g(R, Q)$ and $\beta$ can be determined based on a simulation process. Monte Carlo technique has been used for a simulation to determine the reliability index $\beta$ numerically.
4. Monte Carlo Simulation Method

In this paper, reliability analyses have been achieved through the Monte Carlo method that has used digital computers to generate pseudo-random sampling for variables of dimensions, loads, elastic modulus, and yield stress using Matlab codes.

The method is based on running the model many times as in random sampling. For each sample, random variates are generated on each input variable; computations are run through the model yielding random outcomes on each output variable. Since each input is random, the outcomes are random [16]. The method may be described as a means of solving problems numerically in mathematics, physics, and other sciences through sampling experiments [1].

In each simulation experiment, the possible values of the input random variables \( x = x_1, x_2, \ldots, x_n \) are generated based on predefined distribution and parameters. Then the values of the response variable, \( y \), are determined through the performance function \( y = g(x) \) at the samples of input random variables. In this manner, a set of samples for the response variable \( y \) would be available for the subsequent statistical analyses to estimate the characteristics of the response variable \( y \) [17].

Monte Carlo simulation provides a common feasible way to determine the reliability index or the probability of failure. It is applicable to linear and nonlinear limit state function [12].

The problem to be simulated may have a probabilistic or deterministic form. In the probabilistic form, the actual random variable or function appearing in the problem is simulated, whereas in the deterministic form an artificial random variable or function is first constructed and then simulated [4]. In this paper, the frame can be classified as a deterministic form problem where the stiffness, strength response functions have been determined from the strength of the material and the design of steel structures.

5. Reliability Analysis Using Simulation

Probability failure of a structural element can be calculated based on the amount of convergence function probability distribution of two random variables, Strength (R) and Load (Q). A key point in solving reliability with Monte Carlo simulation methods is the generation of series of random variables for the probability density of each variable of limit state function and failure probability is written as below [8]:

\[
P_f = \frac{\text{Number of trials for } g(x) \leq 0}{N}
\]

It is obvious that the approximation is more realistic with more samples [13].

Although the Monte Carlo method is seen as effective and is widely used in research for reliability, there is a problem with Monte Carlo sampling is that if the probability of failure is a small value such that the structural design where the allowable probability of failure is in the range \((10)^{-5}\) [18], a large number of samples are needed in order to predict this accurately, causing a sharp increase in required cost and time. The other solution is to use algorithms that generate more random numbers near the tail. These kinds of algorithms mostly use a technique to change the dispersion of random numbers in order to generate more random numbers in a certain angle or a specific area such as the tail region [19]. One of these algorithms provided by Far & Wang, 2016 which is known as the approximation of the Monte Carlo sampling method for reliability analysis of structures. A simple algorithm was proposed to estimate low failure probabilities using a small number of samples in conjunction with the Monte Carlo method [20].

The proposed algorithm shown in Figure 2 approximated the failure probability to an acceptable level of accuracy equivalent to the estimation provided by the Monte Carlo method using 20000 random numbers [20]. That has been adopted to estimate the \( P_f \) for deflection and moment limit state for mid-span girder and elastic stability for the frame.

![Figure 2. The proposed approximation flowchart](image)

*Figure 2. The proposed approximation flowchart*
6. System Reliability

Most structures consist of systems of interconnected components and members. When considering the system reliability, it is important to recognize that the failure of a single component may or may not cause the failure of structure [21]. There are three idealized types of structural systems. In a series system, in a parallel system, and hybrid or combined systems. Figure 3 show examples of series and parallel systems.

- **Series systems**

  In this system, the failure of one member leads to immediate failure of the entire system. And it is sometimes referred to as the weakest link system because the failure of the system corresponds to the failure of the weakest element in the system. Probability system failure \( P_f \) represented by the probability of element failure \( P_{fi} \): 

  \[
  P_f = 1 - \prod_{i=1}^{n} (1 - P_{fi}) 
  \]

- **Parallel systems**

  All the members must fail before the system fails:

  \[
  P_f = \prod_{i=1}^{n} P_{fi} 
  \]

- **Hybrid (Combined) Systems**

  Many structures can be considered as a combination of series and parallel systems. Such systems are referred to as hybrid or combined systems [14]. The portal frame considers as hybrid system.

7. Random Variables with Their Statistical Parameters

The statistical characteristics for all parameters that considered in this study have been illustrated in the tables below which they gathered from the review of the literature. Kala, et al. 2009 studied the randomness in cross section dimensions for hot rolled steel sections as illustrated in Table 1 [22], the probability density function that represents the variations in this samples is Normal distribution [22] and [23]. Table 2 present the statistical characteristics for other parameters.

<table>
<thead>
<tr>
<th>Random variables</th>
<th>Relative Mean</th>
<th>Relative Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section depth ( h )</td>
<td>1.0009</td>
<td>0.0044233</td>
</tr>
<tr>
<td>Section width ( b )</td>
<td>1.0139</td>
<td>0.009868</td>
</tr>
<tr>
<td>Web thick. ( t_1 )</td>
<td>1.0540</td>
<td>0.039053</td>
</tr>
<tr>
<td>Flange thick. ( t_2 )</td>
<td>0.9927</td>
<td>0.045859</td>
</tr>
</tbody>
</table>
Table 2. Statistical characteristics for parameters from different sources.

<table>
<thead>
<tr>
<th>Random variables</th>
<th>Mean/ Nominal</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>COV</th>
<th>Distribution type</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point dead load (F_D) kN</td>
<td>1.03</td>
<td>___</td>
<td>___</td>
<td>0.08</td>
<td>Normal</td>
<td>[24]</td>
</tr>
<tr>
<td>Point live load (F_L) kN</td>
<td>1</td>
<td>___</td>
<td>___</td>
<td>0.1</td>
<td>Gumbel</td>
<td>[25]</td>
</tr>
<tr>
<td>Yield strength (F_y) MPa</td>
<td>___</td>
<td>327.50</td>
<td>24.56</td>
<td>0.07</td>
<td>Lognormal</td>
<td>[26]</td>
</tr>
<tr>
<td>Modulus of Elasticity (E) MPa</td>
<td>0.993</td>
<td>___</td>
<td>___</td>
<td>0.034</td>
<td>Normal</td>
<td>[7]</td>
</tr>
<tr>
<td>Girder span m</td>
<td>1</td>
<td></td>
<td></td>
<td>0.07</td>
<td>Lognormal</td>
<td>[13]</td>
</tr>
</tbody>
</table>

This variation can be described based on parameters and statistical distributions indicated in Table 1 and Table 2. Based on these variations and statistical characteristics, Matlab random number generator has been used in this paper to generate a sample for these parameters that have been adopted in subsequent calculations.

8. Case Study of Portal Frame

8.1. Proposed Structural Sections

After check the portal frame sections which they found adequate for the requirements of [27] and assumed braced laterally, the portal frame indicated in Figure 4 has been used in this study. Four concentrated loads have been applied their values shown in Figure 4. The live loads are equal to 25, 12.5 kN and 150, 75 kN are simulate the superimposed load.

![Figure 4. Portal frame with dimensions, sections details, and applied loads](image)

8.2. Limit States Samples

Two samples group are needed for frame components, the first is demand or loads samples, and the second is the capacity. These samples have been generated with respect to uncertainties in their parameters which illustrated in Table 1 and Table 2. As Matlab software has been adopted for this issue.

- **Demand samples**

  The demand samples are represented by deflection, the moment for the girder in addition to moment and axial force for the column. Assume these samples are obtained due to uncertainties in applied loads. The random load values have been generated for 100 values. The frame has been analysis in Abaqus software and by rerun the model for 100 times, the gathered samples have been exported to Matlab to present this data in histogram forms and calculate the statistical characteristics where \(M, \sigma, s, B1,\) and \(B2\) represent mean, variance, standard deviation, coefficient of skewness, and coefficient of kurtosis respectively which indicated in Figure 5 through Figure 8. The \(\chi^2\) test has been used to show that the lognormal probability density function can be adopted for these histograms with coefficients of variance equal to 0.047, 0.048, 0.052, 0.048, and 0.043 for deflection, moment girder, shear force, moment column and axial force samples.
Figure 5. Histogram and statistical characteristics for vertical deflection

Figure 6. Histogram and statistical characteristics for girder moment

Figure 7. Histogram and statistical characteristics for column moment values
Figure 8. Histogram and statistical characteristics for column axial force values

- **Capacity samples**

  The capacity samples for the frame components have been randomly generated using Matlab code. These samples include the maximum allowable deflection and the moment capacity for the girder. Regarding the columns, they include moment and axial compression capacities. These samples with a size of 100 have been generated due to variation in cross-section dimensions, girder span, yield stress, and modulus of elasticity, which their statistical characteristics illustrated in Table 1 and Table 2. These samples presented in histograms in Figure 9 through Figure 12. From the histograms, a lognormal distribution null hypothesis has been adopted and verified using the $\chi^2$ goodness of fit test by Matlab. Therefore, these samples are lognormal probability density function with coefficients of variance equal to 0.064, 0.057, 0.055, 0.053, and 0.040 for max allowable deflection, moment girder, shear force, moment column, and column axial force capacities respectively.

Figure 9. Histogram and statistical characteristics for Max. Allowable deflection
Figure 10. Histogram and statistical characteristics for girder plastic moment sample

Figure 11. Histogram and statistical characteristics for column moment capacity

Figure 12. Histogram and statistical characteristics for column axial force capacity
For the case of elastic stability, the portal frame presented in Figure 13 has been considered which subjected to proportional loads for linear perturbation analysis in Abaqus at the same position of applied loads. Other variables including girder span and columns height have been assumed constant. The frame has been analysed to determine the elastic buckling capacity sample, Eigen values, with changing the aforementioned parameters as input analysis variables using a Matlab code. These data represent the Resistance sample $P_{cr}$, for the stability with statistical properties mentioned in Table 3.

The second sample is the demand or the Load sample $P$. It is represented by the required axial force for the column. The buckling limit state can be defined as [28]:

$$g = P_{cr} - P$$  

If $g < 0$ the column is unstable and represents failure case, otherwise it is stable.

### Table 3. Statistical characteristics for elastic buckling capacity sample

<table>
<thead>
<tr>
<th>Random variables</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>COV</th>
<th>Distribution type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistance sample $P_{cr}$ (kN)</td>
<td>14095</td>
<td>44.671</td>
<td>0.003</td>
<td>Normal</td>
</tr>
</tbody>
</table>

#### 8.3. Reliability Analysis for Frame Components

As shown in Section 8.2 and through using the Monte Carlo simulation method, the generated samples for the demand and capacity indicate no failure at the limit state. It essential to use a larger sample size which is difficult to achieve and required long run time. Therefore, there is a need to use the Approximated Monte Carlo method which had been illustrated in Figure 2 for serviceability and ultimate limit state of the girder in addition to elastic stability limit state for the frame.

Based on the random sample 100 in size, the deflection, moment and stability limit state, LSF, has been drawn versus the standard normal variable, $Z_i$, as indicated in Figure 14 through Figure 16 respectively. The LSF and $Z_i$ have been fitted. Then the equation intercept has been determined to indicate the failure probability. The computed $P_f$ have been summarized in Table 4.
Unfortunately, the approximated method is inapplicable for the column that is adequate when located inside the interaction diagram. To overcome this difficulty, the statistical characteristics of the generated sample have been used to generate a large sample size of 20000 using Matlab.

The axial force bending moment interaction equations specified in (AISC 360, 2010) represents the capacity of a beam-column [7]. Equation 7 has been used in the reliability analysis for the column to indicate that values greater than one represent failures cases.

\[
\frac{P_r}{2P_c} + \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0
\] (7)

A large sample with a size of 20000 has been randomly generated based on the statistical properties of the input parameters. After that, the cases where ratio may be greater than 1 can be counted.

As shown above, the probability of failure for girder and column equal to zero. This may interpret in terms of the applied load magnitude that is very small compared to the failure load.

<table>
<thead>
<tr>
<th>Limit state function</th>
<th>( P_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection limit state</td>
<td>( 1.1 \times 10^{-11} )</td>
</tr>
<tr>
<td>Moment limit state</td>
<td>( 3.2 \times 10^{-37} )</td>
</tr>
<tr>
<td>Stability limit state</td>
<td>0</td>
</tr>
<tr>
<td>Column interaction equation</td>
<td>0</td>
</tr>
</tbody>
</table>
8.4. Case Study for the Frame under Higher Load Value

The failure probability, $P_f$, and the reliability index, $\beta$, for the frame have been re-estimated with the load mean values increase to 60, and 30 kN for live load. While the superimposed load as 200, and 400 kN. As it dealt with the same frame sections and considered the same randomness for the same parameters, the statistical characteristics of Section 8.2 have been used to generate pseudo-random samples. The probability of failure and the reliability index have been presented in Table 5.

<table>
<thead>
<tr>
<th>Limit state function</th>
<th>$P_f$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection limit state</td>
<td>0.9944</td>
<td>-2.54</td>
</tr>
<tr>
<td>Moment limit state</td>
<td>2.9987e-10</td>
<td>6.19</td>
</tr>
<tr>
<td>Stability limit state</td>
<td>0</td>
<td>--------</td>
</tr>
<tr>
<td>Column</td>
<td>0.0094</td>
<td>2.35</td>
</tr>
<tr>
<td>Frame ultimate limit state</td>
<td>8.836e-5</td>
<td>3.75</td>
</tr>
</tbody>
</table>

As illustrated in Table 5, to transform from the member analysis achieved in above to a system reliability analysis for the ultimate limit state function, the frame can be represented as a hybrid system where the overall failure occurs if both columns fail or if the girder fails. Thus, the two columns behave as a parallel subsystem, while both of them are in series with the girder [14]. As the column is identical so that they have the same failure probability and it equal to 0.0094. Hence, for the combined system, the probability of failure using Equation 5 and Equation 6 is equal to $8.836 \times 10^{-5}$ and the reliability index $\beta$ is equal to 3.75. Popov (1990), explained that the $\beta$, for routine applications on order of 3 is considers appropriate [29], therefore $\beta$ for ultimate limit state is appropriate while for deflection is not considers appropriate.

9. Conclusion

In this paper, the Monte Carlo based method and the Approximate Monte Carlo method have been used to estimate the reliability of a portal frame. It has been shown that these methods provide good estimates of the member reliability with a moderate computational effort. The failure probability $P_f$, results indicate that the deflection limit state is more sensitive than other limit state functions for randomness in the considered parameters. The elastic stability of the frame is the least sensitive one. And the variation in applied load is more significant than other random parameters, this due to higher uncertainties in the load samples especially for the live load.

It has been pointed out that the use of Monte Carlo methods for system reliability analysis has several very attractive features, the most important being that the failure criterion is usually relatively easy to check almost irrespective of the complexity of the system and the number of basic random variables.

Ultimate limit state failure probability of the whole frame is dependent on the failure probability for the girder and the columns. It is greater than the member failure probability,$P_{fi}$, for the girder and equal to failure probability of the columns parallel subsystem. From a statistical point of view, the probabilities of failure for elements in the connected joint have been assumed as independence events. From the theory of structure, these probabilities of failure are correlated for an indeterminate structure similar to the considered portal frame.

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11. Conflicts of Interest

The authors declare no conflict of interest.

12. References


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Development of Traffic Volume Forecasting Using Multiple Regression Analysis and Artificial Neural Network

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Abstract

The purpose of this study is to develop a model for traffic volume forecasting of the road network in Anamorava Region. The description of the current traffic volumes is enabled using PTV Visum software, which is used as an input data gained through manual and automatic counting of vehicles and interviewing traffic participants. In order to develop the forecasting model, there has been the necessity to establish a data set relying on time series which enables interface between demographic, socio-economic variables and traffic volumes. At the beginning models have been developed by MLR and ANN methods using original data on variables. In order to eliminate high correlation between variables appeared by individual models, PCA method, which transforms variables to principal components (PCs), has been employed. These PCs are used as input in order to develop combined models PCA-MLR and PCA-RBF in which the minimization of errors in traffic volumes forecasting is significantly confirmed. The obtained results are compared to performance indicators such $R^2$, MAE, MSE and MAPE and the outcome of this undertaking is that the model PCA-RBF provides minor errors in forecasting.

Keywords: Traffic Volume; Forecasting Model; Multiple Regression Analysis; Artificial Neural Network; Principal Component Analysis.

1. Introduction

Transport planning requires the use of demographic and social economic variables in order to estimate traffic volume forecasting for a particular country or region [1]. In recent years traffic volume has been increasing by an annual average of 4.13% in the main road network of Anamorava region, causing a decrease in the service level and resulting in longer travel times, in a decrease of road safety etc. [2]. Road traffic plays an important role in this region because it is the one connection to the country and through its community trips are carried out. This increase has a direct impact in traffic volume forecasting which can be done through forecasting methods such as: econometric regressions, travel-demand modelling and neural network modelling [3].

Many researchers have dealt with the development of models to traffic volume forecast. Morf and Houska (1958) [4] have developed a model to forecast the traffic in rural areas in the State of Illinois (USA) using Multiple Regression Analysis (MLR) method. Tennant (1975) has developed a model for the assessment of traffic volumes in rural area in developing countries including some socio-economic variables, using land and principles of traffic generation in the region of Mali in Kenia. MLR method is used in order to find variables with higher impact which has been the employment followed by vehicle ownership [5].
Then, Neveu (1982) has developed a number of models involving elasticity parameters in MLR in order to forecast traffic volumes as Annual Average Daily Traffic (AADT) for different roads category. Variables included in the model are: population, number of households, vehicle ownership and employment [6]. This model has been improved by Fricker and Saha (1987) increasing the number of variables in order to forecast traffic volume in the rural roads of Indiana State (USA). The traffic volume has been considered as a dependent variable, whilst 13 variables have been taken as independent at the level of country and region [7].

Varagoulī et al. (2005) have developed a model to forecast by MLR method taking into consideration some independent variables which affect the travel demand of the prefecture of Xanthi in Northern Greece [8].

Pupavac (2014) has developed two models to forecast traffic volume on Croatian motorways using econometric methods involving five independent variables and two other dependent variables [9]. Semeida (2014) has developed models according to MLR and Generalized Linear Modelling (GLM) in order to forecast traffic demand for countries with low number of populations, the case of Port Said Governorate in Egypt. He has concluded that GLM model provides the best results in forecasting the number of trips [10].

Nevertheless, methods based on MLR have their defects because dependencies between variables are given in linear form. Thus, with the intention to overcome non-linearity in last decades, neural network has been used in the field of traffic and transport engineering applying various algorithms [11]. ANN has the strong ability to approximate the function and through them the non-linearity between variables and historic traffic data can be reduced compared to other methods [12].

Adamo (1994) has developed a model to estimate AADT using Artificial Neural Network (ANN) and MLR, concluding that the ANN has slightly outperformed the MLR approach [13]. Sharma et al. (2000) have developed models for traffic volume forecast according to traditional methods and ANN in interstates roads with high-volume in Minnesota. The given research is extended to the low-volume roads. By comparing them, it has been found out that ANN provides better results [14].

Tang et al. (2003) have used adapted time-series, neural network, nonparametric regression, and Gaussian maximum methods in order to develop models for traffic volume forecasting by day of the week, by month and AADT for the entire year 1999. The research has been completed using traffic data for the period 1994-1998 in Hong Kong [15]. Duddu and Pulugurtha (2013) have developed a model using statistical methods and ANN taking into account demographic principles in order to estimate link-level AADT based on characteristics of the land use, in the city of Charlotte, North Carolina [16].

Islam et al. (2018) has applied ANN methods and support vector machines (SVM) to estimate AADT based on variables: road geometry, existing counts and local socio-economic data, applying various algorithms for supervised learning of ANN [17]. Park et al. has applied Radial Basis Function (RBF) neural network for traffic volumes forecasting in a freeway. The obtained results show that RBF gives suitable function and it requires less time for calculations [18].

Zhang et al. (2007) have used a combination based on Principal Component Analysis (PCA) and Combined Neural Network (CNN) for short-term traffic flow forecasting. With the transformation of variables using PCA method, Principal Components (PCs) have been used as input data for CNN enabling dimensional reduction of input variables and the size of CNN network. The results according to this approach have been much better than the typical Error Back-Propagation neural network (BPNN) with the same data [19].

Doustmohammadi and Anderson (2016) develop the models that can accurately estimate AADTs within a small or medium sized community. Variables that uses these models are a combination of roadway and socio-economic factors within a quarter-mile buffer of the desired count location. These models were tested and validated to accurately predict across different communities of similar size to support AADT estimation on desired roadways in different communities [20].

Raja et al. (2018) develop a model using linear regression using known AADTs and collection of socio-economic and location variables as a means to estimate the AADT. This model relied on five independent variables nearby population, number of households in the area, employment in the area, population to job ratio and access to major roads. This model is use to estimate traffic volume on low-volume rural and local roads for 12 counties in Alabama [21].

Khan et al. (2018) develops AADT estimation models for different roadway functional classes with two machine learning techniques: Artificial Neural Network (ANN) and Support Vector Regression (SVR). The models aim to predict AADT from short-term counts. The comparison reveals the superiority of SVR for AADT estimation for different roadway functional classes over all other methods [22].

Fu et al. (2016) develops an alternative and low-cost approach for estimating annual average daily traffic values (AADTs) and the associated transport emissions for all road segments in a country. This is achieved by parsing and processing commonly available information from existing geographical data, census data, traffic data and vehicle fleet
data. It was found that AADT estimation based on a neural network performs better than traditional regression models [23].

In this study the model for traffic volume forecasting in Anamorava region is developed. Initially, the current status of traffic volumes in this region has been determined using PTV Visum software, which uses data on traffic volumes as an input. The model is developed according to MLR and ANN methods including 12 independent original variables. In order to develop a model with better performance, respectively to have less errors in forecasting, PCA, in which original variables are transformed in non-correlated PCs, is employed. Those PCs are afterwards used as an input for development of model according to combined PCA-MLR and PCA-ANN methods. In each one of the four methods some significant models have been found, but, based on statistical analysis only the best ones have been selected. Furthermore, comparing those models according to performance indicators, it has been found out that the best model for traffic volume forecasting has resulted to be the one according to PCA-RBF method. The current model is accomplished according to parameters in given region and it can be used in practice.

2. Materials and Methods

This section provides methodology and processing stages for the development of the model for forecasting traffic volume by flow chart as presented by Figure 1.

![Flow chart of the developed models](image-url)

**Figure 1. The flow chart of the developed models**
2.1. Study Area

The development of the model for traffic volume forecasting has been carried out in Anamorava region, which is situated in the Peninsula Balkan and South East of Kosovo, Figure 2. The given region includes six municipalities (Gjilan, Vitia, Kamenica, Parteș, Kllokot, and Ranillug) with an area of 1331 km² [24]. The road network of this region consists of national, regional and local roads. The two national road represent major transportation links between the capital city of this region Gjilan with the other regions of Kosovo and neighbor’s municipality of Kosovo. Also, in Figure 2 shows the map of traffic flow measurement stations (automatic and manual) for this study.

![Figure 2. Anamorava region and its current road network](image)

2.2. Data Collection

In order to develop a model for traffic volumes forecasting, an overview of traffic load distribution on the road network is required. In this regard, data were collected for one work day (15.05) as well as weekend day (21.05) during the period of time 07.00 a.m until 19.00 p.m in May of 2016 with an intention not to require application of a weekly nonlinear coefficient of trips. Traffic counting is accomplished manually (MTC) in eight locations as well as automatic counting (ATC) which took place at four locations (1-Slivovo, 2-Sojevo, 3-Ranillug and 4-Pasjan), with former being suitable for the application of forecasting methods as presented in this paper, Figure 1. There are 11523 interviews conducted based on face-to-face method which consists of 19.43% of the total flow of 59317 vehicles. After the research, counting and interviewing was done using the MS Exel program. Using the ratio between counting and interviewing, it is possible to find traffic volume for 12 h.

Converting traffic from 12h into 24h has been done by employing related correction coefficient gained $K_e, K_{int}$ and $K_{con}$ which shows traffic volumes as AADT [25]. The final origin-destination (O-D) matrix is established by processing and interconnecting counting and interviews of traffic participants for the period of time 24h, by Equation 1.

$$OD_{matrix(24h)} = VOL_{12} \cdot K_e \cdot K_{int} \cdot K_{con} \text{ (vehicle/24h)}$$

(1)

Where $OD_{matrix(24h)}$ – is the origin-destination matrix of trips realized by vehicle in time interval 24 hour, $VOL_{12}$ – is the number of vehicles counting in 12 hours interval, $K_e$ – is the passenger car space equivalent, $K_{int}$=$VOL_{12}/I_{12}$ is the coefficient of interview calculated by number of vehicles counting ($VOL_{12}$) and number of interview ($I_{12}$) realized in 12 hours interval and $K_{con}$=$VOL_{24}/VOL_{12}$ is the converting coefficient of traffic volume from 12 hours to 24 hours.

Once the description of traffic volumes has been made through modelling at PTV Visum, then it was done comparing the results with data count for each location separately. In the beginning there was a discrepancy, but with the application of the balancing process which is based on the production and assign by equilibrium method using the TFlowFuzzy algorithm it was achieved that these discrepancies would achieve satisfactory values [26]. Calibration was carried out with GEH test application [27]. Referring to the results achieved, it is seen that the 8 locations where they were taken for analysis, 7 or 86% of them fulfil the condition defined by GEH <5, as presented in Table 1.
Table 1. Summary of GEH test indicators

<table>
<thead>
<tr>
<th>Evaluation aggregate</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>GEH: Avg.</td>
<td>2</td>
</tr>
<tr>
<td>GEH≤5.0</td>
<td>86%</td>
</tr>
<tr>
<td>Deviation: Avg.</td>
<td>3%</td>
</tr>
<tr>
<td>Deviation: Avg. weighed</td>
<td>4%</td>
</tr>
</tbody>
</table>

Once this matrix is imported to PTV Visum software, the development of the macro model is enabled through which the generation and the distribution of traffic volumes conducted for unit of vehicle category on current road network of this region is obtained, Figure 3 [28]. The model includes 373 nodes, 948 connections and 13 zones.

In order to develop a model for traffic volumes forecasting, initially, 12 demographic and socio-economic variables which have an impact in traffic demand have been identified. Afterwards, the data-set is established for these variables in time historic format for the period 2004-2016 [29], through which it was enabled to establish dependence with traffic volumes. The data-set related to the traffic volumes is established for four locations in which automatic counting are static.

2.3. Modelling Methods

In order to develop the model for traffic volume forecasting MLR and ANN methods as well as combined methods PCA-MLR and PCA-ANN are employed [30].

2.3.1. Multiple Regression Analysis

Multiple regression analysis is a statistical method used to investigate the relation between variables based on mathematical model that called regression model. MLR explains dependent variable \( y \) as the result of changing in \( k \), independent variables \( (x_1, x_2, ..., x_k) \) to certain size and direction, where \( i \) represents number of years, \( k \) is the number of independent variables. A schematic form of MLR method is shown in Figure 4.

Figure 3. Traffic volume distribution on current main road network

Figure 4. Schematic form of MLR method
The general form of MLR is expressed by Equation 2 [33]:

\[ y_i = \beta_0 + \beta_1 \cdot x_{i1} + \beta_2 \cdot x_{i2} + \ldots + \beta_k \cdot x_{ik} + \epsilon_i \]  

(2)

Where, \( y_i \): is dependent variable, \( \beta_0 \): is intercept, \( \beta_1, \beta_2, \ldots, \beta_k \): are regression coefficients, \( x_{ik} \): are independent variables and \( \epsilon_i \): is error associated during regression.

Here it is important to verify whether two or more independent variables are strongly correlated, known as the multicollinearity phenomenon. If this happens then it has taken measures for its elimination, because it affects negatively to the predictive ability of the model. Different techniques are used to eliminate it. As a technique that is commonly used is that "stepwise" because during model selection includes statistical indicators VIF and DW [31]. This technique works on the principle of adding and removing variables in each iteration.

In case we have only two alternative models with a level of significance \( p<0.05 \), choosing one of them as best is done by evaluating parameters according to the standard error by applying the "Sum of Squares" method to the prediction. Testing is done through various statistical indicators R, \( R^2 \), Adjusted R, ANOVA through test F, t-test, residual analysis etc. The final selection of one of the significant models is done by selecting the minimum value that ANOVA gives under the Fisher F-test. But in cases where a significant number of models is greater than two but finite, for selecting the best model, are using selection criteria as well: Akaike Information Criterion (AIC), Amemia Prediction Criterion (APC), Mallow’s Prediction Criterion (Cp), Schwarz Bayesian Criterion (SBC) [32].

2.3.2. PCA-MLR method

The PCA method is part of the multivariate statistical nonparametric method, through which the elimination of the high correlation between the initial variables (multicollinearity phenomenon) and improved the predictive ability of the model, namely reducing error in prediction. This method is used to summarize the information collected by several observed variables that are strongly correlated with each other and by reducing them to a smaller number of factors or by forming a new data set that contains a number of principal components (PCs). These obtained PCs are non-correlated and they get linear weight like a combination of original variables and they are also used as input for MLR method. Moreover, PCA method relies on three basic steps: estimation of suitability of data, extracting main components and rotation of vectors. In order to justify PCA method it is necessary to verify suitability data through tests according to Kaiser-Meyer-Olkin (KMO>0.5) and Bartlett Test (\( p<0.05 \)) [33]. The next step is to extract PCs, which are obtained by calculating the eigen values of the matrix. They PCs which have eigen values greater than 1 (eigen>1) should be taken into consideration during the construction of the model where the order is made going from the largest variance to the smallest. The last step is the rotation of factors through which new factors can be acquired and interpreted.

The analysis through PCA-MLR enables a combination of PCA and MLR methods in order to establish mutual relation between dependent variable \( y_i \) and PCs which are obtained as the result of multiplying original independent variables \( x_{ik} \) by eigenvectors. A schematic form of PCA-MLR is shown in Figure 5.
Where, $\beta_0$ is intercept, $\beta_1, \beta_2, ..., \beta_k$ are the regression coefficients, $PC_1, PC_2, ..., PC_n$ are basic components, $u_e$ is the error associated during regression.

### 2.3.3. Artificial Neural Network

ANN shows high level interface adaptation of non-linear processing neuron elements for parallel processing of data through simple way. ANN method avoids detailed mathematical analysis and it is used to overcome non-linearity which is present through input and output variables used to develop the model [34]. This method is also used to learn, to adjust, to generalize, to investigate and to reproduce linear and nonlinear relation between variables etc. [35]. There are several variants of ANN method. Based on processing information they are classified into: feed forward (Single Layer Perceptron, Multilayer Perceptron, Radial Basis Network) and back forward (Competitive Networks, Kohonen’s SON, Hopfield Network, ART models). One variant is RBF neural network which works in feed-forward error-back propagation network, and it is widely used. RBF has a simple topology and it consists of three layers: input layer, hidden layer and output layer, Figure 6. Nodes contained in hidden layer have non-linear transfer function with radial base, while nodes in output layer have linear transfer function.

RBF is suitable for application in estimating problems where limited data exist and overtraining should be avoided. Generalization at the vicinity of center groups is maintained by scaled nature of transferring functions. Information on RBF network is distributed in local area and as a result only some weights are modified in each iteration in the training process. For its application, a normalization of data is done for every input variable through Equation 4:

$$Z = \frac{x - \mu}{\sigma}$$

(4)

Where, $x$ is the observed value for every variable, $\mu$: is the average value of variables, $\sigma$: is the standard deviation. Input layer consist of some nodes in which data processing is not done but only input vector is applied ($x=x_1, x_2, ..., x_k$). Hidden layer consists of $N_h$ ($h=1, 2, ..., n$). The number of $N_h$ in hidden layer is equal to the number of centers accumulated used to training data. Group centers represented by vectors $\mu_j$ ($1 < j < N_h$) have been obtained using fuzzy $c$-means algorithm technique [36].

![Figure 6. Architecture of RBF model](image)

Connection weight connecting input nodes $i$ to hidden layer $j$ is equal to $\mu_{ij}$ which correspond to the $i^{th}$ component of $\mu_j$ vector. An output of the hidden node $j$ is determined according to Gauss transformation function, Equation 5:

$$\Phi_j = \exp\left(-\frac{||x - \mu_j||^2}{2 \cdot \sigma_j^2}\right)$$

(5)

Where, $x$: is the input vector for neuron, $\mu_j$: is the centric value of basis node $j$ in hidden layer, $||x - \mu_j||$: is the Euclidean distance between a center vector and the set of data points, $\sigma_j^2$: is the variance of the function for each of the centers ($j$) or range of influence of the Gaussian function from centers $\mu_j$ and is calculated according to Equation 6 [36]:

$$\sigma_j = \frac{1}{3N_h} \sum_{i=1}^{N_h} \|\mu_j - \mu_i\|^2, \ 1 \leq j \leq N_h$$

(6)
In Equation 6, factor $\sigma_j$ is equal to 1/3 of the value of the number of nodes $N_0$ and average distance from group centers. Connection of nodes from hidden layer $j$ in output node layer is defined by its weight $\lambda_j$. The value $y$ in the output of network is shown by Equation 7:

$$y = \sum_{j=1}^{N_h} \phi_j \cdot \lambda_j$$

(7)

The weights $\lambda_j$ are calculated after getting minimal error of network between the value at output $y$ and the desired value $y_d$ based on the data-set of network training. Furthermore, in order to train network and find out weights $\lambda_j$, the problem should be solved through unconstrained optimization method Equation 8:

$$\text{Minimize} E(\lambda) = \sum_{i=1}^{N} ||y_i' - y_i'||$$

(8)

Where: $N$: is the total number of cases of training sample, $\lambda_j$ are weights which connection of nodes from hidden layer $j$ in output node layer, $y_i'$ -value in output layer, $y_i'$ - desired value in output layer. In order to solve the problem of minimization of error descent gradient algorithm is used as shown in [37].

2.3.4. PCA-RBF Method

The combination of PCA and RBF methods, PCA-RBF method is combined through which there is possibility to get the relation between the dependent variable $y_i$ and obtained uncorrelated PCs as input variables in RBF [38]. For this reason, we initially apply the PCA method as a preprocessor to the neural network according to the RBF method to eliminate high correlation between original variables $x_{ik}$. It is known fact that PCA operates with the data that function in the linear form, while that of the ANN for data that have a nonlinear form. The idea here is when they are used together, to have the opportunity to effectively cover the linear and nonlinear part of the forecast. Moreover, the combination according to this approach will minimize the complexity of problem of training in network. The way of functioning is presented in Figure 7.

![Figure 7. Architecture of PCA-RBF model](image)

3. Results and Discussions

In order to develop a model to forecast the traffic volume there is the necessity to fulfil some preconditions such as variance, normality tests, graphical method and description of variables. The analysis starts with variance of testing of data-set for dependent variable $y_i$ for locations such Slivove, Sojeva, Ranillug and Pasjan. In order to estimate the variance on traffic volumes for above mentioned locations Levene test is used through which homogeneity is verified for the level of significance (Sig.>0.05), as presented in Table 2.

| Table 2. Variance test of homogeneity for dependent variable $y_i$ |
|-------------------|---|
| Levene Statistic  | 0.867 |
| Sig.              | 0.429 |

This test with the value 0.867 it means that the variance of homogeneity between locations has approximate value. For the given locations, testing of normality of data is done through Shapiro-Wilk test, in which the value of the coefficients $B$ next to each location has turned to be at significant level Sig>0.05, as presented in Table 3.
Table 3. Normality test for dependent variable $y_i$

<table>
<thead>
<tr>
<th>Type of road</th>
<th>Location</th>
<th>Number of observed</th>
<th>Kolmogorov Test</th>
<th>Shapiro Wilk Test</th>
<th>$\beta$</th>
<th>Sig.</th>
<th>Normality</th>
</tr>
</thead>
<tbody>
<tr>
<td>National</td>
<td>All locations</td>
<td>39</td>
<td>0.112</td>
<td></td>
<td>0.200</td>
<td>0.05</td>
<td>fulfilled</td>
</tr>
<tr>
<td>National</td>
<td>Slivovo</td>
<td>13</td>
<td>0.823</td>
<td>0.013</td>
<td>0.05</td>
<td></td>
<td>fulfilled</td>
</tr>
<tr>
<td>National</td>
<td>Sojevo</td>
<td>13</td>
<td>0.888</td>
<td>0.093</td>
<td>0.05</td>
<td></td>
<td>fulfilled</td>
</tr>
<tr>
<td>National</td>
<td>Ranillug</td>
<td>13</td>
<td>0.914</td>
<td>0.207</td>
<td>0.05</td>
<td></td>
<td>fulfilled</td>
</tr>
<tr>
<td>Regional</td>
<td>Pasjan</td>
<td>13</td>
<td>0.826</td>
<td>0.014</td>
<td>0.05</td>
<td></td>
<td>fulfilled</td>
</tr>
</tbody>
</table>

This result proves that there is no necessity to do transformation of data according to any function (log, sqrt etc.). Therefore, in order to simplify the problem, based on results of two tests which was fulfilled in general only Slivove location, which represents other locations, is treated.

Apart from this, graph method is also used to verify the dependency of any independent variable $x_{ik}$ to the dependent variable $y_i$ in which it results in variable $x_{12}$ (number of vehicles registered in Anamorava level), which shows a tendency and a dependency which is more sustainable with dependent variable and expectation for increase for the period of time 2004 – 2016.

In this regard, a statistical description is accomplished for the presentation of variables which take place in developing model to traffic volume forecasting, are shown in Table 4.

Table 4. Basic descriptive statistics for variables

<table>
<thead>
<tr>
<th>Name of variable</th>
<th>Symbol</th>
<th>N</th>
<th>Min</th>
<th>Max</th>
<th>Mean</th>
<th>Std.Dev</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic Volume</td>
<td>$y_1$</td>
<td>1</td>
<td>6325</td>
<td>10439</td>
<td>7449</td>
<td>1240</td>
</tr>
<tr>
<td>State Population</td>
<td>$x_1$</td>
<td>12</td>
<td>1786282</td>
<td>1891906</td>
<td>1846303</td>
<td>37473</td>
</tr>
<tr>
<td>State Household</td>
<td>$x_2$</td>
<td>12</td>
<td>278915</td>
<td>338618</td>
<td>311062</td>
<td>17863</td>
</tr>
<tr>
<td>State Employment</td>
<td>$x_3$</td>
<td>12</td>
<td>236181</td>
<td>340911</td>
<td>290450</td>
<td>33631</td>
</tr>
<tr>
<td>State Vehicle Registration</td>
<td>$x_4$</td>
<td>12</td>
<td>179157</td>
<td>336942</td>
<td>249102</td>
<td>52192</td>
</tr>
<tr>
<td>Consumer Price Index</td>
<td>$x_5$</td>
<td>12</td>
<td>77</td>
<td>101</td>
<td>90</td>
<td>9</td>
</tr>
<tr>
<td>Gross Domestic Product</td>
<td>$x_6$</td>
<td>12</td>
<td>3006100</td>
<td>5984900</td>
<td>4431790</td>
<td>1072114</td>
</tr>
<tr>
<td>Per Capita Income</td>
<td>$x_7$</td>
<td>12</td>
<td>1763</td>
<td>3356</td>
<td>2507</td>
<td>562</td>
</tr>
<tr>
<td>Gasoline Price</td>
<td>$x_8$</td>
<td>12</td>
<td>0.840</td>
<td>1.160</td>
<td>1.015</td>
<td>0.027</td>
</tr>
<tr>
<td>Region Population</td>
<td>$x_9$</td>
<td>12</td>
<td>240502</td>
<td>254723</td>
<td>248583</td>
<td>5045</td>
</tr>
<tr>
<td>Region Household</td>
<td>$x_{10}$</td>
<td>12</td>
<td>48999</td>
<td>51442</td>
<td>50504</td>
<td>685</td>
</tr>
<tr>
<td>Region Employment</td>
<td>$x_{11}$</td>
<td>12</td>
<td>32720</td>
<td>43692</td>
<td>37302</td>
<td>3983</td>
</tr>
<tr>
<td>Regional Vehicle Registration</td>
<td>$x_{12}$</td>
<td>12</td>
<td>29031</td>
<td>53806</td>
<td>39419</td>
<td>8514</td>
</tr>
</tbody>
</table>

The development and the estimation of significant model is done by employing the above-mentioned methods and the SPSS software. In the following, the results of the model according to each method are presented and discussed.

3.1. Results by MLR

The outcome of the matrix of correlation shows that each $k$ of independent variable ($x=x_1, \ldots, x_{12}$) in relation to dependent variable $y_i$ fulfills condition that the values of correlation coefficient are ($r>0.8$). Apart from this, it is obvious that independent variables have high correlation (dependency) and as a result there is multi-co linearity phenomenon. This phenomenon has been overcome by using stepwise technique in SPSS software which is functioning according to forward and backward method in order to add and remove variables, resulting in 14 significant candidate models at the p<0.05 level and two non-significant models with p>0.05. Summarized results are given in Table 5.
Table 5. Model summary

<table>
<thead>
<tr>
<th>Model</th>
<th>Predictor variables</th>
<th>R</th>
<th>R²</th>
<th>Adjusted R²</th>
<th>Std. Error</th>
<th>Sig.F Change</th>
<th>Durbin Watson</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>x₁</td>
<td>0.723⁺</td>
<td>0.523</td>
<td>0.480</td>
<td>894.36176</td>
<td>0.005</td>
<td>0.557</td>
</tr>
<tr>
<td>2</td>
<td>x₂</td>
<td>0.738⁺</td>
<td>0.545</td>
<td>0.503</td>
<td>873.93719</td>
<td>0.004</td>
<td>0.805</td>
</tr>
<tr>
<td>3</td>
<td>x₃</td>
<td>0.866⁺</td>
<td>0.751</td>
<td>0.728</td>
<td>646.79968</td>
<td>0.000</td>
<td>1.197</td>
</tr>
<tr>
<td>4</td>
<td>x₄</td>
<td>0.925⁺</td>
<td>0.856</td>
<td>0.843</td>
<td>491.76565</td>
<td>0.000</td>
<td>1.113</td>
</tr>
<tr>
<td>5</td>
<td>x₅</td>
<td>0.803⁺</td>
<td>0.645</td>
<td>0.613</td>
<td>771.38572</td>
<td>0.001</td>
<td>0.568</td>
</tr>
<tr>
<td>6</td>
<td>x₆</td>
<td>0.890⁺</td>
<td>0.792</td>
<td>0.773</td>
<td>590.74995</td>
<td>0.000</td>
<td>0.658</td>
</tr>
<tr>
<td>7</td>
<td>x₇</td>
<td>0.901⁺</td>
<td>0.813</td>
<td>0.796</td>
<td>560.79044</td>
<td>0.000</td>
<td>0.725</td>
</tr>
<tr>
<td>8</td>
<td>x₈</td>
<td>0.461⁺</td>
<td>0.212</td>
<td>0.141</td>
<td>1149.63385</td>
<td>0.113</td>
<td>0.363</td>
</tr>
<tr>
<td>9</td>
<td>x₉</td>
<td>0.723⁺</td>
<td>0.523</td>
<td>0.480</td>
<td>894.34210</td>
<td>0.005</td>
<td>0.557</td>
</tr>
<tr>
<td>10</td>
<td>x₁₀</td>
<td>0.663⁺</td>
<td>0.440</td>
<td>0.389</td>
<td>969.14285</td>
<td>0.013</td>
<td>0.617</td>
</tr>
<tr>
<td>11</td>
<td>x₁₁</td>
<td>0.888⁺</td>
<td>0.789</td>
<td>0.770</td>
<td>594.69288</td>
<td>0.000</td>
<td>1.063</td>
</tr>
<tr>
<td>12</td>
<td>x₁₂</td>
<td>0.933⁺</td>
<td>0.871</td>
<td>0.859</td>
<td>465.53011</td>
<td>0.000</td>
<td>1.338</td>
</tr>
<tr>
<td>13</td>
<td>x₅, x₃</td>
<td>0.947⁺</td>
<td>0.897</td>
<td>0.877</td>
<td>435.47191</td>
<td>0.000</td>
<td>1.822</td>
</tr>
<tr>
<td>14</td>
<td>X₅, x₈</td>
<td>0.938⁺</td>
<td>0.880</td>
<td>0.856</td>
<td>470.61794</td>
<td>0.000</td>
<td>1.385</td>
</tr>
<tr>
<td>15</td>
<td>X₅, x₉</td>
<td>0.940⁺</td>
<td>0.883</td>
<td>0.860</td>
<td>464.40321</td>
<td>0.000</td>
<td>1.364</td>
</tr>
<tr>
<td>16</td>
<td>X₅, x₁₁</td>
<td>0.919⁺</td>
<td>0.844</td>
<td>0.812</td>
<td>537.03697</td>
<td>0.000</td>
<td>1.627</td>
</tr>
</tbody>
</table>

a. Predictors: x₁, x₂, x₃, x₄, x₅, x₆, x₇, x₈, x₉, x₁₀, x₁₁, (x₅,x₈), (x₅,x₉), (x₅,x₁₁).
b. Dependent variable: y.

Based on the results of the table above it is seen that model 12 gives the maximum value of the determination coefficient (R²) of 0.871 (what it means 87.1% of the variance of the response variable (ds) explained by the model). Adjusted R²=0.859 shows that 85.9% of variation of dependent variable is explained by the variation of independent variables. In this regard, the sustainability of the model according to Durbin-Watson (DW=1.338) test has also been verified, which is not within the interval of auto correlation of 1.5 to 2.5.

However, as we have already explained above, only through this coefficient we are not sure whether we have found the best model because we have some significant models. Therefore, we apply the selection criteria where the results are presented in Table 6.

Table 6. Summary of selection criteria

<table>
<thead>
<tr>
<th>Model</th>
<th>df1</th>
<th>df2</th>
<th>Akaike Information Criterion</th>
<th>Amenia Prediction Criterion</th>
<th>Mallows' Prediction Criterion</th>
<th>Schwarz Bayesian Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>11</td>
<td>178.527</td>
<td>0.650</td>
<td>2.000</td>
<td>179.657</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>11</td>
<td>177.927</td>
<td>0.621</td>
<td>2.000</td>
<td>179.056</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>11</td>
<td>170.101</td>
<td>0.340</td>
<td>2.000</td>
<td>171.231</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>11</td>
<td>162.976</td>
<td>0.197</td>
<td>2.000</td>
<td>164.106</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>11</td>
<td>174.681</td>
<td>0.484</td>
<td>2.000</td>
<td>175.811</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>11</td>
<td>167.745</td>
<td>0.284</td>
<td>2.000</td>
<td>168.874</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>11</td>
<td>166.391</td>
<td>0.256</td>
<td>2.000</td>
<td>167.521</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>11</td>
<td>185.055</td>
<td>1.074</td>
<td>2.000</td>
<td>186.185</td>
</tr>
<tr>
<td>9</td>
<td>1</td>
<td>11</td>
<td>178.527</td>
<td>0.650</td>
<td>2.000</td>
<td>179.656</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>11</td>
<td>180.615</td>
<td>0.763</td>
<td>2.000</td>
<td>181.745</td>
</tr>
<tr>
<td>11</td>
<td>1</td>
<td>11</td>
<td>167.917</td>
<td>0.287</td>
<td>2.000</td>
<td>169.047</td>
</tr>
<tr>
<td>12</td>
<td>1</td>
<td>11</td>
<td>161.551</td>
<td>0.176</td>
<td>2.000</td>
<td>162.681</td>
</tr>
<tr>
<td>13</td>
<td>2</td>
<td>10</td>
<td>160.576</td>
<td>0.164</td>
<td>3.000</td>
<td>162.271</td>
</tr>
<tr>
<td>14</td>
<td>2</td>
<td>10</td>
<td>162.594</td>
<td>0.192</td>
<td>3.000</td>
<td>164.289</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>10</td>
<td>162.249</td>
<td>0.187</td>
<td>3.000</td>
<td>163.944</td>
</tr>
<tr>
<td>16</td>
<td>2</td>
<td>10</td>
<td>166.027</td>
<td>0.250</td>
<td>3.000</td>
<td>167.722</td>
</tr>
</tbody>
</table>

Based on these results the model 12 is selected as the best because it gives smaller value by selection criteria, compared with other candidate models. To compare the goodness of fit of model 12 and intercept only-model (i.e., mean value of the response variable) is used the ANOVA test as shown in Table 7.

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In this regard, it has also been proven the significance of model according to ANOVA, F-test ($F=74.157$) with reliability value 95% respectively the $p$-value is less than 0.01 ($p=0.000<0.05$), which shows that the alternative hypothesis is verified $H_a$: where at least one of the independent variables is statically significant and different from zero while rejecting the hypothesis $H_0$: at 99% confidence level.

Also, by results in Table 8 for $t$-test ($t=8.611$) it is clear that variable $x_{12}$ is a significant variable with the value of reliability 95% that $p$-value is less than 0.05 and it may be used to develop a model in order to forecast the traffic volumes. Apart from this, the value of the coefficient (VIF=1<10) shows that multi-collinearity does not exist.

Therefore, MLR model expressed through variables $x_{12}$ is presented by Equation 9:

$$Y = 2091.432 + 0.136 \cdot x_{12}$$

(Equation 9)

Equation 9 shows that the variable $x_{12}$ has a positive impact in traffic volumes, which means that by increasing the level of motorization in the level of Anamorava region the value of dependent variable “traffic volume” is also increasing. The weakness of this model is that as a consequence of high correlation, as a result number of independent variables drastically falls in model development.

3.2. Results by PCA-MLR

In order to include more variables in developing the model combination of methods PCA and MLR is used. In order to apply this method, some necessary conditions are fulfilled according to Kaiser-Meyer-Olkin test (KMO=0.720>0.5) and Bartlett’s test of Sphericity Sig.(p<0.05), as presented in Table 9.

In order to find the eigenvalues associated to each factor, are necessary to use the phases: before extraction, after extraction and rotation. Before the phase of extraction there are 12 linear components identified within the data set. By using PCA method all original variables are grouped in two factors which are named $PC_1$ and $PC_2$ by eigenvalue bigger than 5% with the variability about 96%, while another PCs which have the value approaching to zero has been removed from the model, because it shows high correlation between each other and means there is presence of multicollinearity.

Based on matrix components, are proved that exists simple correlation or have higher values (higher than 0.9) between all original variables and new PCs. Thus, variables $x_{11}$, $x_4$, $x_7$, $x_6$, $x_{12}$, $x_3$ and $x_5$ have high impact in $PC_1$, while variables $x_1$, $x_9$, $x_8$, $x_{10}$ and $x_2$ have more high impact in $PC_2$. While multiplying scores of coefficients (eigenvalues) and values of original variables obtained score $s$ for each PCs as presented by Equations 10 and 11.

$$PC_1=0.030 \cdot x_1+0.044 \cdot x_2+0.156 \cdot x_3+0.172 \cdot x_4+0.066 \cdot x_5+0.125 \cdot x_6+0.136 \cdot x_7-0.113 \cdot x_8+0.030 \cdot x_9-0.05 \cdot x_{10}+0.147 \cdot x_{11}+0.200 \cdot x_{12}$$

(Equation 10)

$$PC_2=0.177 \cdot x_1+0.150 \cdot x_2-0.037 \cdot x_3-0.064 \cdot x_4+0.120 \cdot x_5+0.023 \cdot x_6+0.02 \cdot x_7+0.379 \cdot x_8+0.177 \cdot x_9+0.224 \cdot x_{10}-0.019 \cdot x_{11}-0.118 \cdot x_{12}$$

(Equation 11)
These PCs are used as independent variables in MLR analysis to determine all significant PCs, which could be used in the model. Using stepwise procedure in SPSS software the results are obtained according to this hybrid method as shown in Table 10.

<table>
<thead>
<tr>
<th>Model</th>
<th>R</th>
<th>R²</th>
<th>Adjusted R²</th>
<th>Std Error</th>
<th>D. Watson</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.948a</td>
<td>0.899</td>
<td>0.879</td>
<td>431.58135</td>
<td>1.486</td>
</tr>
</tbody>
</table>

The results show that the dependent variable has high correlation of obtained PCs and it is qualified as independent variables and $R^2=0.899$ show that 89.9% of dependent variable is explained by non-correlated PCs. Adjusted $R^2=0.879$ show that 87.9% of variation of dependent variable is explained by variation of PCs. Also, it has also been verified the sustainability of the model according to Durbin-Watson ($DW=1.486$) test which is not within the interval of auto-correlation 1.5 to 2.5.

<table>
<thead>
<tr>
<th>Model</th>
<th>SumSquares</th>
<th>Df.</th>
<th>Mean Square</th>
<th>F</th>
<th>Sig.(p&lt;0.05)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>16592424.14</td>
<td>2</td>
<td>8296212.07</td>
<td>44.540</td>
<td>0.000b</td>
</tr>
<tr>
<td>Residual</td>
<td>1862624.62</td>
<td>10</td>
<td>186262.46</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>18455048.76</td>
<td>12</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

It has also been verified by results in Table 11 that the significance of the model according to ANOVA, associated with F-test ($F=44.540$) the value of reliability 95% respectively the $p$-value is less than 0.01 ($p=0.000<0.05$), which indicates that the alternative hypothesis is verified $H_a$: where at least one of the independent variables is statically significant and different from zero while rejecting the $H_0$: hypothesis at 99% confidence level.

<table>
<thead>
<tr>
<th>B</th>
<th>Std Error</th>
<th>t</th>
<th>Sig.(p&lt;0.05)</th>
<th>Tolerance</th>
<th>VIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant</td>
<td>7449.308</td>
<td>119.699</td>
<td>62.234</td>
<td>0.000</td>
<td></td>
</tr>
<tr>
<td>PC1</td>
<td>1516.286</td>
<td>205.210</td>
<td>7.389</td>
<td>0.000</td>
<td>0.369</td>
</tr>
<tr>
<td>PC2</td>
<td>-473.241</td>
<td>205.210</td>
<td>-2.306</td>
<td>0.044</td>
<td>0.369</td>
</tr>
</tbody>
</table>

In this regard, in Table 12 are presented results by t-test ($t=7.389$ and $t=-2.306$) that shows the condition of significance is fulfilled for two PCs with values of reliability 95% $Sig.(p=0.000<0.05)$ and $Sig.(p=0.044<0.05)$ used for development of model for traffic volumes forecasting. Apart from this, the value of coefficient ($VIF=2.713<10$) shows that multicollinearity does not exist. Thus, the model is expressed through $PC_1$ and $PC_2$ components by Equation 12:

$$y = 7449.308 + 1516.286 \cdot PC_1 - 473.241 \cdot PC_2$$

(12)

By Equation 12, shows that $PC_1$ component has a positive impact whilst $PC_2$ component has a negative impact in traffic volumes. The advantage of this method is that reduces multicollinearity phenomenon and the complexity of model development, while the disadvantages of this method are that in interpretation between original variables and PCs also the model is developed only in a linear form.

### 3.3. Results by RBF

In order to develop a model with optimal network, some models of neural network of RBF type according to "trial and errors" technique are designed which differ in numbers of neurons in hidden layer, the way of functioning is activated as well as rules of learning. Before beginning with the training of data-set, normalization of input variables is done. Data in data-set gained in 13 observations (13 years) has been completed in randomly and for training purposes 10 observations are taken or 76.9 %, while for testing 3 observations or 23.1%.

Training data in data set is used to develop a model and to find out weights, while the data of testing are used to find errors and to prevent overtraining in the training process. The determination of the number of neurons of RBF network is done according to automatic way. The number of neurons at input layer is 12, while the number of neurons at hidden layer is 5, whilst the number of neurons at output layer is 1.

Neurons at output layer mean forecasted traffic volumes. Activation function “softmax” is applied to connect number of neurons from hidden layer to the output layer. For output layer, “identity” is used as activation function. In order to
calculate an error, “sum of squares error” is applied. In Table 13, are given the overall results of the model associated with the description of the error generated by the neural network together with the ratio (percentage) of inaccurate predictions during the training and testing phases.

<table>
<thead>
<tr>
<th>Table 13. Model Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Training</strong></td>
</tr>
<tr>
<td>Relative Error</td>
</tr>
<tr>
<td>Training Time</td>
</tr>
</tbody>
</table>

| **Testing**              | **Sum of Squares Error** | **0.038** |
| Relative Error           | 0.104                     |

Dependent variable $y$.

Table 13 shows that the error in training is (SSE=0.049) which shows the significance of model for forecasting. Error when doing testing is (SSE=0.038) which means that the model is not overtraining. Furthermore, percentage in inaccuracy in forecasting training is 0.011 (or 1.1%) while in testing is 0.104 (or 10.4%). The dependency of values observed with those forecasted is provided in Equation 13:

$$y = 15.96 + 1.01 \cdot x$$  \tag{13}$$

From Equation 13, $R^2=0.986$ is gained and it shows that observations of forecasted values have high dependence and that this is suitable model for forecasting.

3.4. Results by PCA-RBF

In order to minimize error in forecasting combined method PCA and RBF is used. Similarly, like the PCA-MLR method, required tests for PCA are given in Table 9. In this case obtained PCs are used as an input in neural network of RBF type. In order to prove the validity of PC the sample of data-set is divided in two parts: training and testing one. Before starting with training of data-set, normalization of PC is done using Equation 4. Data in the data-set cover 13 observations (13 years) chosen in random bases, for training 9 observations are taken which is 69.2%, while for testing purposes 4 observations are taken which is 30.8%. Selection of optimal network architecture is done in automatic way selecting 2 neurons in input layer, 5 neurons in hidden layer and 1 neuron in output layer. The neuron in output layer means also variable on traffic volume $y$.

Activation function “softmax” is applied at neurons in hidden layer, while “identity” is used at neurons in output layer. In order to calculate an error, “sum of squares error” is applied. The summary of obtained results related to the inaccuracies of the forecasts are shown in Table 14.

<table>
<thead>
<tr>
<th>Table 14. Model Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Training</strong></td>
</tr>
<tr>
<td>Relative Error</td>
</tr>
<tr>
<td>Training Time</td>
</tr>
</tbody>
</table>

| **Testing**              | **Sum of Squares Error** | **0.010** |
| Relative Error           | 0.026                     |

Dependent variable $y$.

Results shown in Table 14 show that an error in training is (SSE=0.005), while an error in testing is (SSE=0.010) which means that the model is stronger and it is not over trained. Furthermore, the percentage in inaccuracy in forecasting training is 0.001 (or 0.1 %) while at testing is 0.026 (or 2.6%). Dependency of values observed with the ones forecasted is reflected in Equation 14:

$$y = 49.21 + 1.01 \cdot x$$  \tag{14}$$

From Equation 14, $R^2=0.997$ is gained and it shows the measured observations with the values of forecasting with high dependency and the model is suitable for forecasting.

The results gained according to four methods mentioned above in graph way are shown in Figure 8. In Figure 8, a black curve shows the data measured for the traffic volume by automatic counting, also, volumes of traffic forecasted by (MLR-orange, PCA-MLR-green, RBF-blue, PCA-RBF-red) methods are shown. Comparing the results in the graph shows that the volumes measured with the ones forecasted on traffic are approximate by all four methods, but more sustainable results are given by PCA-RBF method; the red curve in red complies with the black one.
4. Performance Comparisons of Models

In order to determine which one of the four methods used provides higher accuracy in developing a model on traffic volume forecasting, models are established and compared with each other based on performance indicators such as: $R^2$, MAE, RMSE and MAPE [33]. In order to calculate those indicators, forecasted data are included $F_i$ as well as the ones measured $A_i$. The obtained results are shown in Table 15.

<table>
<thead>
<tr>
<th>Performance Indicators</th>
<th>MLR</th>
<th>PCA-MLR</th>
<th>RBF</th>
<th>PCA-RBF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adjusted $R^2$</td>
<td>0.99839</td>
<td>0.99980</td>
<td>0.99874</td>
<td>0.99996</td>
</tr>
<tr>
<td>MAE</td>
<td>361</td>
<td>106</td>
<td>336</td>
<td>50</td>
</tr>
<tr>
<td>RMSE</td>
<td>428</td>
<td>150</td>
<td>370</td>
<td>70</td>
</tr>
<tr>
<td>MAPE</td>
<td>4.25</td>
<td>1.38</td>
<td>4.46</td>
<td>0.70</td>
</tr>
</tbody>
</table>

The results of indicators show that models relied on neural network compared to the ones based on linear regression provide lower error in forecasting because models of neural network are expressed in the form of non-linearity. This was expected due to the relation between independent variables and traffic volumes in the form on non-linearity. The best model is offered by PCA-RBF method because an error in forecasting is lower. Therefore, the model developed according to this method is suggested to be used for traffic volumes forecasting for Anamorava region.

5. Conclusion

This study the development of the model for traffic volume forecasting in Anamorava region is presented. Current state of distribution of traffic volumes is expressed by employing PTV Visum software which serves as starting point to develop forecasting model. With the intention to develop a model, 12 variables with an impact in generating traffic volumes are identified, establishing a data-set in the form of time series. Applying these variables and employing methods relied in regression and neural network, certain significant models are developed.

Based on the analysis of the results of these models, it has been found out that the model based on neural PCA-RBF is the best one because it provides lower value error in traffic volume forecasting. Therefore, this model can be used also in the preparation of the transport planning strategy for this region.
6. Acknowledgment

The authors expressed thanks to the students at the Department of Traffic and Transport Engineering/University of Prishtina for their assist in conducting counting and interviews in main road network of Anamorava region.

7. Conflict of Interests

The authors declare no conflict interest.

8. References


Major Parameters Affect the Non-Liner Response of Structure Under Near-Fault Earthquakes

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Abstract

Near-fault ground motion can be identified by the presence of a predominant long duration pulse in the velocity traces mainly due to directivity effect. This pulse exposes the structure to high input energy at the beginning of the earthquake which leads to a higher response in comparison with the ordinary ground motions. This paper investigates 79 earthquake records with different properties to achieve three goals: the first aim is to compare between the linear and nonlinear response of SDOF systems under near-fault and far-fault earthquakes. While the second objective is to examine the parameters that control the characteristics of near-fault earthquakes. Two factors have been studied which is PGV/PGA ratio and pulse period. Finally, the seismic code provisions related to the near-fault earthquakes were evaluated in term of the elastic acceleration response spectrum, the evaluation is adopted for American Society of Civil Engineers code ASCE 7 and Uniform Building Code UBC. The results lead to the following conclusions: with respect to a specific PGA, the near-fault earthquake imposed higher response in comparison with far-field earthquakes. The near-fault earthquakes become severe as the PGV/PGA and pulse period increase. The interested seismic codes can cover the actual behavior based on the average response of a certain amount of data, while it may become non-conservative relative to an individual record.

Keywords: Near-Fault Earthquake; Pulse Period; PGV/PGA; Strength Reduction Factor; Response Spectrum.

1. Introduction

Near-field earthquake identified by limited frequency and high amplitude pulse with a long duration that may or may not appear in the acceleration time history but it is significantly obvious in the velocity traces. This kind of ground motions put the structures under high input energy at the starting of earthquake due to the effect of two phenomena called directivity effect and fling step effect. The directivity effect occurs when the fault rapture travels toward the site at a velocity very close to the shear wave velocity. This will expose the structures to high amplitude with long duration pulse that extremely affects the structural response. Conversely, the backward directivity effect happens when the fault spread apart from the site, such effect can produce a long duration ground motion traces with small amplitude [1]. On the other hand, the fling step effect takes place when the two sides of the fault moved relative to each other. This movement happened in a manner that causes permanent displacement on the tectonic plate which leads to one side pulse in the velocity time history and step pulse in the displacement traces [2].

Due to the special properties of the near-fault earthquakes, it deserves a deeper discussion to clarify all the mysterious points related to this phenomenon. Many conclusions regard the structural response under near-fault earthquakes have been discussed in previous attempts. Chopra and Chintanapakdee (2001) diagnosed the changes in the spectral regions

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due to near-fault effect. It is pointed out that, in case of near-fault motions, the acceleration and displacement sensitive regions become much wider while the velocity sensitive region becomes narrower in comparison with far-field motions. This effect increases the range of structures that respond in a stiff manner under near-fault earthquakes [3]. The linear and nonlinear response of frame buildings that subjected to the near-fault earthquake is studied by Alavi and Krawinkler (2004) [4]. The results showed that, for a structure with a period longer than pulse period, the yielding starts at high stories, however as the base shear strength drop, the ductility demands shift to the bottom stories. While for a structure with a period less than pulse period, a large ductility demand always occurs in the lower stories. Furthermore, the difference in response of shear wall building under near-fault and far-fault motions is illustrated by Heydari and Mousavi (2015) [5]. It is observed that the relative displacement due to near-field records is twice that obtained from far-field records. Additionally, the relative displacement of the building under the near-field effect increases as the ratio of the structural period to the pulse period becomes larger.

Alhan and Sürmeli (2015) examined the validity of near-source factors that adopted in the UBC code by studying the response of 3, 8 and 15 story building subjected to near-field ground motions. It is concluded that the provisions of seismic codes without near-source factors underestimate the behavior of the buildings [6]. However, the examined near-source factors need more modifications to produce a better response estimation. Hosseini et al. (2017) discussed the suitability of the seismic code provisions that should ensure life safety performance level for the reinforced concrete buildings under near-fault earthquakes. Three-component nonlinear time history analysis conducted for a set of buildings. He found that the performance of some cases reached beyond the life safety level and some of them even collapsed. So it is concluded that the code requirements need to be improved and consider the high intensity of the vertical ground motion component in the near-source earthquakes [7]. Talebi Jouneghani et al. (2017) points out that the linear response of high-frequency structures to near-fault earthquakes is less than nonlinear behavior of the same buildings, since the near-ground motions have low-frequency content and the nonlinearity decreases the frequency of the structure [8]. Kohrangi et al. (2018) investigate how the shape of the acceleration response spectrum for pulse-like motions affects the response of the buildings. By examining a set of earthquakes with and without pulse-like which has equivalent acceleration response spectrum shapes. He noticed that the severity of the near-fault earthquakes cannot be predicted only by the shape of the spectrum and it’s important to investigate deeply the pulse properties which have the major effect on the intensity of the ground motion [9].

It is noted that the previous attempts are dealing with a relatively limited number of ground motions to investigate a specific case study which normally leads to conditional results. Hence it is important to perform comprehensive investigation including a wide range of near-fault earthquakes that has different characteristics such as fault mechanism, site condition, PGA and magnitude. This paper investigates 79 earthquake records with different properties to achieve the following topics: discussing the response of elastic and inelastic SDOF systems under near-fault and far-fault excitations. Investigating the effect of pulse period and PGV/PGA on the response of SDOF systems. Assessing the seismic code provision with regard to near-fault earthquakes based on elastic response spectrum. An overview of the steps of the research methodology can be seen in Figure 1.

2. Selected Ground Motions

A set of 79 earthquakes was selected from pacific earthquake engineering research center (PEER) ground motion database [10], including 74 near-fault and 5 far-fault ground motions. The magnitudes of near-fault earthquakes range from 5 to 7.6 with the closest distance to the fault plane, not more than 30 km. These earthquakes are classified as pulse-like ground motion according to a technical report for PEER ground motion database [11]. On the other hand, the selected far-fault earthquakes namely without pulse, have a magnitude range from 5.7 to 7.28 and located at a distance not less than 87 km from the fault plane. Tables 1 and 2 show the details of selected near-field and far-field earthquakes respectively.

3. Software Analysis

The Dynamic analysis is conducted by Prism version 1.0.2 which is a seismic analysis application dealing with only single degree of freedom systems and can perform the following functions: modifying the earthquake records, adopting the Newmark integration method to conducting the time history analysis by employing a several type of hysteresis patterns, and also, the ability of determining the elastic and inelastic response spectrum. The results of Prism software are verified with the results obtained by Sap2000. Both programs produce identical results in linear and nonlinear states.

The examined SDOF system has a range of vibration periods starting from 0.02 sec and limited by 10 sec with step 0.02 sec, all these vibration periods chosen under 5% damping ratio. The material nonlinearity is represented by the variety of the stiffness. While the damping is assumed to be constant through elastic and inelastic stages. The bi-linear hysteretic model is adopted to represent the nonlinear behavior, in which the post to pre-yielding stiffness ratio set to zero. Whereas the ductility factor, defined as the ratio of the maximum inelastic displacement to yield displacement, selected to be 2.
Figure 1. Flowchart of research methodology

Table 1. Near fault earthquakes

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Station</th>
<th>$M^1$</th>
<th>$R_{rup}$ km</th>
<th>PGV/PGA sec</th>
<th>$T_p$ sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Fernando, US</td>
<td>Pacoima Dam (upper left abut)</td>
<td>6.6</td>
<td>0.18</td>
<td>0.10</td>
<td>0.16</td>
</tr>
<tr>
<td>Coyote Lake, US</td>
<td>Gilroy Array #6</td>
<td>5.7</td>
<td>0.11</td>
<td>0.1</td>
<td>0.2</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>Aeropuerto Mexicali</td>
<td>6.5</td>
<td>0.14</td>
<td>0.22</td>
<td>0.23</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>Agrarias</td>
<td>6.5</td>
<td>0.32</td>
<td>0.22</td>
<td>0.3</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>Brawley Airport</td>
<td>6.5</td>
<td>0.23</td>
<td>0.23</td>
<td>0.4</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>EC County Center FF</td>
<td>6.5</td>
<td>0.32</td>
<td>0.32</td>
<td>0.33</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>El Centro - Meloland Geot. Array</td>
<td>6.5</td>
<td>0.32</td>
<td>0.32</td>
<td>0.33</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>El Centro Array #10</td>
<td>6.5</td>
<td>0.32</td>
<td>0.32</td>
<td>0.33</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>El Centro Array #11</td>
<td>6.5</td>
<td>0.12</td>
<td>0.12</td>
<td>0.18</td>
</tr>
<tr>
<td>Imperial Valley-06, US</td>
<td>El Centro Array #3</td>
<td>6.5</td>
<td>0.18</td>
<td>0.18</td>
<td>0.52</td>
</tr>
<tr>
<td>Location</td>
<td>Station Name</td>
<td>Peak Acceleration</td>
<td>Peak Veloc.</td>
<td>Peak Disp.</td>
<td>Year</td>
</tr>
<tr>
<td>----------------------------------</td>
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$E_{max} = \frac{1}{2} k S_{d, max}^2$  \hspace{1cm} (1)

$V_{max} = k S_{d, max}$  \hspace{1cm} (2)

Where $E_{max}$ is the maximum strain energy, $k$ is the stiffness of the SDOF structure, $S_{d}$ is the displacement response spectrum and $V_{max}$ is the maximum base shear force.

Equations 1 and 2 imply that the displacement response spectrum can clearly quantify the maximum strain energy stored in the structure and the maximum base shear force. Hence it is can be noticed that near-fault earthquakes subject the structure to high input energy and produce a higher strength demand in comparison with the far-fault earthquakes. One more point should be mentioned here, that in spite of both near-fault and far-fault excitations have similar PGA, the response of the elastic SDOF systems exhibit larger demands in near-fault case relative to far-fault one. That means, the severity of earthquakes was poorly measured by the PGA, and the structural response is sensitive to the pattern of acceleration variation with time.

4. Linear Response

Earthquake spectrum response can illustrate a comprehensive understanding of the structural behavior with a wide range of vibration periods. Since that each point on the response spectrum curves represents a peak response of the SDOF structure with associated vibration period. This advantage is employed to compare near-fault with far-fault earthquakes in term of the displacement, velocity and acceleration response spectra. To simplify the presentation of results, only five near-fault earthquakes were chosen to compare with the mean response spectrum of the five far-fault earthquakes as shown in Figure 2. All records considered in this research were normalized to satisfy PGA equal to 30% of the gravity acceleration (0.3g).

Figure 2 shows that near-fault earthquakes create a larger response with considerable difference in displacement and velocity spectra while the difference tends to reduce in the acceleration spectra. This gives evidence that the acceleration response spectrum cannot accurately measure the severity of the pulse-like ground motions so that the response spectrum analysis should be avoided and more detailed analysis such as time history is preferable in case of design building to near-fault earthquakes. To demonstrate the obtained results, two relations should be highlighted here [12]:

$E_{max} = \frac{1}{2} k S_{d}^2$  \hspace{1cm} (1)

$V_{max} = k S_{d, max}$  \hspace{1cm} (2)

Where $E_{max}$ is the maximum strain energy, $k$ is the stiffness of the SDOF structure, $S_{d}$ is the displacement response spectrum and $V_{max}$ is the maximum base shear force.
5. Nonlinear Response

The strength reduction factor \( R \), defined as the ratio between the strength required to keep the system in the elastic range \( F_o \) to the yield strength of the system \( F_y \) (\( R = F_o / F_y \)), is used to quantify the strength demand of the inelastic SDOF systems. The value of \( R \) is equal to unity in the linear SDOF system and it is greater than 1 in the nonlinear range, which means that when the strength reduction factor of a structure has a small value beyond the unity, the structure requires a high yielding strength to withstand the earthquake and vice versa.

The SDOF systems are exposed to normalize near fault-and far-fault excitation produced by Chi Chi and Landers earthquakes respectively. Figure 3 illustrates the variation of the strength reduction factor \( R \) with respect to the period of vibration \( T \) of elastic perfectly plastic SDOF systems with ductility factor of 2 under near-fault and far-fault ground motions. It is observed that the strength reduction factor of the near-fault record is less than far-fault which means that for the same PGA and ductility demands the near-fault excitation produce a larger strength demand. Moreover, the strength reduction factor tends to be 1 at very short periods which implies that in spite of the considered ductility factor, there is no reduction in the design strength, while at very long periods the strength reduction factor approaches the value of ductility factor 2. Accordingly, if the long periods structures permitted to undergo through inelastic behavior, then the design strength is significantly reduced corresponding to the selected ductility factor.
into two groups, the first one with PGV/PGA < 0.2 and the second one with PGV/PGA > 0.2 knowing that each set contains 37 ground motion records. These earthquakes are used to study the response of elastic perfectly plastic SDOF systems with ductility factor of 2. Figure 4 shows the mean inelastic displacement against the vibration periods for the two groups of studied earthquakes. It is noted that short period systems insensitive to the variation of PGV/PGA ratio, on the other hand, the long vibration period structures exhibit a significantly larger displacement corresponding to higher PGV/PGA ratio. This results in line with that obtained by Liao et al. (2001) when he examined the effect of PGV/PGA ratio on two buildings with vibration periods 0.78 sec and 1 sec where the longest period building exhibit a larger story drift as the PGV/PGA ratio increase [13]. Conversely, the shorter period buildings not much affected by increasing the PGV/PGA ratio.

6. The Effect of PGV/PGA Ratio

Near-fault motions imposed a relatively high ratio of PGV to PGA, due to the distinct velocity pulse associated with such kind of motions. To investigate the effect of this ratio on the response of inelastic SDOF systems, the set of 74 near-fault earthquakes presented in Table 1 are divided into two groups, the first one with PGV/PGA < 0.2 and the second one with PGV/PGA > 0.2 knowing that each set contains 37 ground motion records. These earthquakes are used to study the response of elastic perfectly plastic SDOF systems with ductility factor of 2. Figure 4 shows the mean inelastic displacement against the vibration periods for the two groups of studied earthquakes. It is noted that short period systems insensitive to the variation of PGV/PGA ratio, on the other hand, the long vibration period structures exhibit a significantly larger displacement corresponding to higher PGV/PGA ratio. This results in line with that obtained by Liao et al. (2001) when he examined the effect of PGV/PGA ratio on two buildings with vibration periods 0.78 sec and 1.4 sec where the longest period building exhibit a larger story drift as the PGV/PGA ratio increase [13]. Conversely, the shorter period buildings not much affected by increasing the PGV/PGA ratio.

7. The Effect of Pulse Period

To discuss how the period of coherent velocity pulse impacts the behavior of the structures, four near-fault earthquakes with different pulse periods (Tp) were selected and examined in term of the inelastic displacement spectrum with ductility factor equal to 2.

The selected earthquakes are Coyote Lake, station Gilroy Array #6; Imperial Valley, station El Centro Array #6, station El Centro Array #8 and Landers, Yermo Fire Station, and their pulse periods are 1.2, 3.8, 5.4 and 7.5 sec respectively. Figure 5 shows the inelastic displacement response spectrum for selected earthquakes.

It is observed that the spectrum has a bell shape around the pulse periods which mean that the inelastic displacement demands increased as the vibration period of the structure approaches the pulse period. So it is essential to keep the
vibration periods of the designed structure as far as possible from the pulse periods of the design earthquake to avoid high response demand. For a comprehensive study, more records need to be evaluated, for that the near-fault ground motions presented in Table 1 are assorted to three ranges of pulse periods: $T_p < 2$ sec, $2 < T_p < 5$ sec, $T_p > 5$ sec. The mean inelastic displacement spectrum with ductility factor of 2 for the three ranges of pulse periods are illustrated in Figure 6. It is clear that the spectrum demand increases as the pulse periods become longer and the peak displacement demands occur around the pulse periods ranges.

![Figure 4. The effect of PGV/PGA ratio of near fault earthquakes on the response of inelastic systems](image)

![Figure 5. The effect of pulse period on the inelastic displacement response spectra](image)
8. Comparison with Seismic Codes

Seismic codes aim to produce a structure which can adequately withstand the deformations and forces due to an earthquake. However, the provisions of these codes cannot totally prevent the damages, but it should protect the structures from collapse during the ground motions. The design response spectrum is an effective tool offered by seismic codes to predict the behavior of structures under seismic loads. Many factors should be considered to derive the shape and amplitude of the design response spectrum such as the magnitude of the earthquake, the shortest distance to the fault plane, the type of soil, and the importance of the structure. Depending on such parameters, each seismic code performed its own seismic hazard analysis to produce a smoothed design response spectrum which should satisfy the requirements of seismic design.

ASCE 7 [14], and UBC [15] codes were selected in this research to discuss the validity of the design response spectrum presented by these codes with respect to near-fault earthquakes. The ASCE 7 code presents two scales for the elastic design response spectrum: one depending on the maximum considered earthquake (MCE) which is rare event assumed to happen once each 2500 years, and the second depends on the design earthquake that occurs once each 500 years. These response spectra are derived with respect to three parameters: soil site class, risk category, and the spectral response acceleration parameters $S_s$ and $S_1$. The evaluating of these parameters entails three steps: first, a proper soil site class should be selected depending on the shear wave velocity at the top 30m of the soil profile. Where the ASCE 7 classified the soil to 6 classes from A to F starting from the strong soil (rock) to weak soil (soft clay). Secondly, the occupancy of the target structure should be specified to choose one of the four risk categories proposed by ASCE 7. Finally, $S_s$ and $S_1$ obtained from a contour map given within ASCE 7 which describe how the MCE affects the short-period structures (0.2 sec.) and long-period structures (1 sec.) respectively.

The ASCE 7 code did not include the near-fault effects clearly in the derivation of the elastic response spectrum, the UBC considers that effect directly by suggesting near-source factors which applies to the area located near the active seismic source. In addition to the soil type, the elastic response spectrum in a near-fault region controlled by two other parameters: the closest distance to the known seismic source and the magnitude of the design earthquake. These factors used to amplify the seismic coefficients $C_a$ and $C_v$ which describe the elastic response spectrum. Since the response spectrum of UBC depending on the magnitude of the earthquakes, the near-fault records presented in Table 1 are classified into 3 boundaries according to the magnitude: $M \geq 7$, $6.5 \leq M < 7$, $M < 6.5$ which identified as group A, B, and C respectively. Then each group is compared with the associated response spectrum proposed by UBC. On the other hand, the ASCE 7 used to compare with 45 near-fault motions that occurred in the United State that implicitly presented Table 1.

Figure 7 presents the elastic response spectrum of UBC with respect to the mean spectrum of the corresponding near-fault record group. The mean plus standard deviation spectrum is also included to indicate how much the average difference of the records from the mean value.

According to the examined data, it is observed that the mean spectrum in harmony with that of UBC, except the spectrum of group A where the response of long duration periods (longer than 4 sec) is underestimated by UBC spectrum. However, the mean plus standard deviation spectrum considerably higher than code spectrum, this difference is observed at the long periods in group A and at the amplitude of the spectrum in group B and C. The maximum difference has an amplification value of 1.88, 1.65 and 1.86 for group A, B and C respectively.
Figure 8 shows the evaluation of ASCE 7 elastic spectrum relative to the mean spectrum of associated earthquakes. In spite of that, ASCE 7 doesn't consider the near-source factors, it has covered the average response spectrum of the examined set of records exceedingly. Once more the mean plus standard deviation spectrum introduces a response higher than the code spectrum with a maximum difference of 1.4 at the short period range. It is concluded that the examined seismic codes spectrum may underestimate the structural response with respect to the individual near-fault earthquakes while it can catch the mean response if several records were considered. This is reasonable because the seismic codes produce the shape and the amplitude of the spectrum depending on a wide range of data.
9. Conclusions

This study examined the behavior of elastic and inelastic SDOF systems under a range of near-fault and far-fault ground motions with different characteristics to get comprehensive results that may be used to predict the response of the multi-degree of freedom structure which has the same vibration period. It is noted that the previous attempts are dealing with a relatively limited number of ground motions to investigate a specific case study which normally leads to conditional results. Hence it is important to perform comprehensive investigation including a wide range of near-fault earthquakes that has different characteristics such as fault mechanism, site condition, PGA and magnitude. In this paper, we investigated 79 earthquake records with different properties. The dynamic analysis results lead to the following conclusions:

- The large displacement response spectrum of the near-fault earthquakes indicates that such kind of ground motions subject the structure to high input energy and produce a high strength demand in comparison with the far-fault earthquakes.
- The acceleration response spectrum cannot accurately measure the severity of the pulse-like ground motions so that the response spectrum analysis should be avoided and more detailed analysis such as time history is preferable in case of design building to near-fault earthquakes.
- The PGA is an inaccurate measurement of the earthquake's intensity. Where the near-fault ground motions produce a higher response in comparison with the far-fault motions for a similar value of PGA.
- It is observed that for the identical PGA value and same ductility demands, the near-fault excitation produces a larger strength demand relative to ordinary earthquakes.
- The strength reduction factor of the very short period structures (stiff structures) tends to be a unity, which means that in spite of the considered ductility factor there is a very small reduction in the strength gains by the nonlinear analysis.
- The PGV/PGA ratio has a significant effect on the structural response especially for a long period structure where the response tends to be larger as the PGV/PGA increases.
- Stiff structures with high frequency and short vibration period are insensitive to the variations of the PGV/PGA ratio.
- The most dominate property in case of pulse-like ground motion is the pulse period relative to the structural period. Where the response demand increased as the structural period approaches the pulse period.
- The actual response spectra of a single near-fault earthquake may exceed the elastic spectra of ASCE7 and UBC codes with significant amplification factor. While it can catch the mean response if several records were considered. This is reasonable because most seismic codes produce the shape and the amplitude of the spectrum based on a wide range of data.
10. Conflicts of Interest

The authors declare no conflict of interest.

11. References


Effect of Sheet Pile Driving on Geotechnical Behavior of Adjacent Building in Sand: Numerical Study

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Abstract

Construction vibration such as sheet pile driving can produce earthborn vibrations which may be leads to problems for the supporting soils and adjacent structures. Vibrations create the stress waves traveling outward from the source through the soil and cause structural damage due to dynamic vibration induced settlement. The main aim of the present research is to study the vibration effect through sheet pile driving technique on the surrounding soil and adjacent structure. A series of plain strain finite element analysis using Plaxis 8.2 dynamic module is run to simulate the installation technique of a sheet pile unit using driving technique (hammer type). The effect of construction stages with different embedded sheet pile depth, sand relative density, and foundation distance from the driving source is also studied. The influence of hammer driving amplitude on the foundation response and excess pore water pressure are presented. The results showed that the increase of both embedment sheet pile depth and hammer efficiency can significantly produce higher excess pore water pressure and foundation settlement. The increase of sand density can also has a great effect in increasing the foundation damage of adjacent structure compared with low sand relative density. The building damage can significantly take place when the driving is closed to foundation.

Keywords: Finite Element; Sheet Pile Driving; Plaxis; Pore Pressure; Dynamic Settlement.

1. Introduction

Building vibrations can generate soil vibration with variation in intensity, which mainly depends upon the source of vibration. Pile driving is mainly used in many applications for geotechnical engineering (i.e. foundation support and etc. Pile-driving is installed typically by use of impact or vibratory hammers.

Ground vibrations due to pile driving are part of a complex process. Vibration is generated from the pile driver to the pile. As the pile interacts with the surrounding soil, vibrations are transferred at the pile-soil interface. The vibration propagates through the ground and interacts with structures, both above ground and underground. The vibration continues into the structure where it may disturb occupants and/or damage the structure.

The vibration waves may cause potential damage of existing building induced by vibration source. More specifically, these vibrations can cause ground settlements and deformations that may lead to differential settlements of foundations. In case of vibratory sheet piling, generation and dissipation of excess pore pressure occurs simultaneously. It has been found that the interim drainage results in a significant decrease in pore pressure generation.

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Sassa and Sekiguchi (1999) executed model tests on the development of excess pore pressure in the seabed. The tests are performed by placing a small scale wave tank in a geo-centrifuge [1].

During the test the wave loading is constant. In the test generation and dissipation of excess pore pressure occurs simultaneously. The test shows that the excess pore pressure first increases. At this stage the generation of pore pressure is in excess of the dissipation. After some time the excess pore pressure decreases. This indicates that the generation of excess pore pressure is decreasing. Two mechanisms can be held responsible for this effect. The first is the densification of the sand due to the dissipating water. The other effect is the change in soil fabric during densification.

The same effect is observed in the tests by Sumer et al. (1999) in a small tank at ISVA, Denmark. In the tests the development of excess pore pressure in a sandy/silty seabed due to wave loading is measured. The excess pore pressure at first rises quickly. Even complete liquefaction is observed in the test. After some time the excess pore pressure decreases. Towards the end of the test the excess pore pressure is completely vanished, although the wave loading remained constant. These model tests show that dissipation of excess pore pressure increases the resistance against liquefaction [2].

Mitchell and Dubin (1986) used a very interesting test device. In the cyclic tri-axial apparatus a column filled with sand is connected to the drainage valve. The column is used to simulate the flow resistance encountered in reality by dissipating excess pore pressure. With this device the amount of dissipation could be controlled [3]. Dissipation of excess pore pressure greatly reduces the generation of excess pore pressures. This effect is therefore to be accounted for when considering the soil behaviour during vibratory sheet piling. For large shear strain amplitudes (typically shear strain amplitudes in excess of 1%) the positive effect becomes a negative effect. Monitoring and control of construction vibrations were studied by a number of researchers e.g. [4-9] to evaluate the different effects of vibration to prevent any possible damage.

Where most well documented papers discuss behaviour of pile under dynamic driving process, on the other hand a few researchers investigate behaviour of the sheet pile. So the dynamic behaviour of soil due to driving the sheet pile needs deep investigation to reach well understanding its effect on ground surface and adjacent building. The main objective of the present paper is to study theoretically the effect of driving technique of sheet pile at different embedded lengths, the sand density and the hammer efficiency on the induced pore water pressure. Also, the study shed the light on the optimum safe distance adjacent to the building foundation to avoid any damage due to such obtained disturbance from pile driving.

2. Mechanism of Wave Propagation

During process of sheet pile driving seismic waves are generated. These waves transmitted through the soil by two mechanisms. First one is the shear waves, which called S-waves, it generated along the contact surface the sheet pile with the surrounding soil due to the relative motion between the sheet pile and the surrounding soil. The sheet pile driven vertically to the soil, which produces compression waves [10-17]. The S-waves spread out from the sheet pile shaft on a conical wave front as shown in Figure 1 [18]. Second type of waves is called P-waves or compression waves. It starts at the tip of the sheet pile as shown in Figure 2 [18]. Compression waves and shear waves travel quickly from sheet pile toe and sheet pile surface, respectively outwards to the ground surface through new waves called Rayleigh waves (R-wave) as shown in Figure 2. The main factors effect on dynamic waves propagation through the soil are the sheet pile depth, the soil stiffness, the soil homogeneity, and the driving energy delivered to the sheet pile [19-23].

![Figure 1. Generation mechanism of shear waves due to soil–pile friction [18]](image)
3. Finite Element Models and Analysis

Finite element analyses of sheet pile driving are carried out using Plaxis 8.2 2D dynamic version. A set of general fixities to the boundary conditions of the problem are considered automatically by the Plaxis program. The Rayleigh damping is considered at vertical boundaries with $\alpha, \beta = 0.01$ in order to resist the Rayleigh waves. While the plastic properties of soil are defined by using material damping, which is defined in Plaxis by Rayleigh ($\alpha$ and $\beta$), where The Rayleigh damping is considered to be object-dependent in material data set to consider the plastic properties of soil during the dynamic analysis in Plaxis. Plaxis models for sheet pile driving are shown in Figure 3. For impact hummers, the analysis was based on three phases plastic (staged construction) and two phases for dynamic analysis (total multipliers). The dimensions of the soil model for pile driving is taken around 50 m in depth and 150 m in width after some mesh experiments.

3.1. Description of the Model

Different parameters used in the Plaxis model are illustrated as shown in Table 1. "Mohr Columb" undrained model is used for modelling sand. The sheet pile in Plaxis is modelled as a linear elastic non porous. The sheet pile wall is modelled using 6-noded elastic plate element with variable depth. Table 1 enlists the properties of the sheet pile wall. Figure 4 shows model deformation and Figure 5 shows the distribution of excess pore water pressure.
3.2. Studied Parameters

In this research the parameters are varied to evaluate their effects on the soil response under the effect of pile driving. Table 2 shows a series of the studied models that were run for the problem under investigation. Nine runs were carried out in order to investigate the effect of different parameters on the surrounding soil. In Table 3 the data of the adjacent building are summarized according to the manual user of Plaxis programme

Table 1. The different parameters used in the models in this research

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Sand 1</th>
<th>Sand 2</th>
<th>Sand 3</th>
<th>Steel Sheet Pile</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight above pheratic line</td>
<td>γ_unsat</td>
<td>17</td>
<td>18</td>
<td>20</td>
<td>72</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Unit weight below pheratic line</td>
<td>γ_sat</td>
<td>19</td>
<td>20</td>
<td>22</td>
<td>—</td>
<td>kN/m³</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E_ref</td>
<td>20000</td>
<td>33000</td>
<td>40000</td>
<td>2×10⁵</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Oedometer modulus</td>
<td>E_oed</td>
<td>20000</td>
<td>33000</td>
<td>40000</td>
<td>—</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Power</td>
<td>m</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Unloading modulus</td>
<td>E_u</td>
<td>75000</td>
<td>99000</td>
<td>120000</td>
<td>—</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>ν</td>
<td>0.3</td>
<td>0.33</td>
<td>0.35</td>
<td>0.15</td>
<td>—</td>
</tr>
<tr>
<td>Friction angle</td>
<td>Ø</td>
<td>30</td>
<td>35</td>
<td>45</td>
<td>—</td>
<td>°</td>
</tr>
<tr>
<td>Dilatancy angle</td>
<td>ψ</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>—</td>
<td>°</td>
</tr>
<tr>
<td>Interface strength reduction</td>
<td>R_inter</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Bending stiffness</td>
<td>EI</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>2.3×10⁶ kN.m²/m</td>
</tr>
</tbody>
</table>
Table 2. Summary of the studied series under investigation

<table>
<thead>
<tr>
<th>Constant parameter</th>
<th>Variable parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy soil with $E=33000$ kN/m$^2$, hammer weight 10ton, distance of the building from the driving source 10m, building width 6m.</td>
<td>$L_{pile} = 20, 15, 8$ m</td>
</tr>
<tr>
<td>Sandy soil with $E=33000$ kN/m$^2$, hammer weight 10ton, building width 8m, $L_{pile}6$m</td>
<td>Distance of the building from the driving source $= 5, 10$ m.</td>
</tr>
<tr>
<td>hammer weight 10ton, distance of the building from the driving source $5$m, building width $8$m, $L_{pile}6$m</td>
<td>$E=20000, 33000, 40000$ kN/m$^2$</td>
</tr>
</tbody>
</table>

Table 3. Material properties of the building (plate properties)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Floors/walls</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Model</td>
<td>Elastic</td>
<td>--</td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>$EA$</td>
<td>$5.106$</td>
<td>kN/m</td>
</tr>
<tr>
<td>Flexural rigidity</td>
<td>$EI$</td>
<td>$9000$</td>
<td>kN/m$^2$/m</td>
</tr>
<tr>
<td>Weight</td>
<td>$w$</td>
<td>$5$</td>
<td>kN/m/m</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>$\nu$</td>
<td>$0.0$</td>
<td>--</td>
</tr>
<tr>
<td>Rayleigh dampers</td>
<td>$\alpha$ and $\beta$</td>
<td>$0.01$</td>
<td>--</td>
</tr>
</tbody>
</table>

4. Results and Discussions

4.1. Effect of Sheet Pile Embedment Lengths on Pore Pressure and Soil Settlement

It has been found that the ground vibration due to pile driving generate from pile toe according to [8], the energy transmission efficacy correctly reflects the vibration emission from the pile to the surrounding soil after [8, 10]. Therefore the effects of pile length on the vibration amplitude and pore pressure consider an important parameter to be discussed. Trial 1 based on pile length of 20 m to illustrate the effect of the maximum embedment length. The pore pressure and soil settlement are taken on different places on the model (at the ground surface, at depth of 2.5,7m in the close corner of the building to the driving source 10 m horizontally, in the middle of the building 13m horizontally and the other corner of the building 16m horizontally respectively).

The Excess of Pore Pressure

![The Excess of Pore Pressure](image)

Figure 6. The pore water pressure values due to driving pile at length of 20m and $D/L = 0.5$
Figure 6 shows the pore pressure values at ratio of $D/L = 0.5$ where $L$ is the pile length and $D$ is the distance of the building from the driving source. It can be noticed that the pore pressure is increased by getting close to the pile toe and the start to be decreased at the ground surface and by getting far horizontally from the pile driving. Trial # 2 at pile length of 15 m and at the same places of measurements with $D/L = 0.67$, it has found that the same trend is given but with higher values of pore pressure as shown in Figure 7. As it is expected in trial # 3 for $D/L = 1.25$, the values of pore pressure is increased by almost the double compared to trial #2, as shown in Figure 8.

![Figure 7. The pore water pressure values due to driving pile at length of 15m and $D/L = 0.67$](image7.png)

![Figure 8. The pore water pressure values due to driving pile at length of 8m and $D/L = 1.25$](image8.png)

For the same condition soil displacement is measured typically in the same places to express soil settlement. It is found that for relatively high density of sand soil settlement considers to be low. Figures 9 to 11 show the values of soil displacement for different cases of pile driving (at different embedment lengths). It can be seen that soil displacement increases by increasing the pile length close to pile toe. At 10 m horizontally from the driving source at the ground surface there is no clear effect of driving energy on settlement values; however there are other parameters could be harmful to the building. Soil settlement is higher than the ground surface and it could be harmful to underground structures, tunnels, and water or gas pipe lines.
Figure 9. The soil settlement values due to driving sheet pile at length of 20m and D/L = 0.5

Figure 10. The soil settlement values due to driving sheet pile at length of 15m and D/L = 0.67

Figure 11. The soil settlement values due to driving sheet pile at length of 8m and D/L = 1.25
4.2. Effect of Different Distances of the Adjacent Building from the Driving Source on Pore Pressure and Soil Settlement

The main goal of this research is to know the safe limits of pile driving procurement to avoid any possible damage to the adjacent building. Focus will be on the effective distance of the adjacent structure which cannot be exceeded. Different distances are used to represent the adjacent building to sheet pile driving and expressed by the term of (W) the other parameters are constant. It is known that the great values of vibration amplitude is close to the driving source and start to attenuate by getting far from the driving source after [10]. To reach the ultimate case a 6m sheet pile (L=6) is used for giving the higher possible vibration amplitude. The first trial for 5m distance from the driving source (W=5) and the ratio W/L=0.83. It can be seen that pore pressure values increased closed to the driving source and exceeded the damage criteria and would cause significant damage to the building. Fig.11 shows the pore pressure values at W/L = 0.83 which consider unsafe ratio and the design engineer should avoid achieving that ratio to protect the adjacent buildings. Fig.12 shows the results of pore pressure measurements at 10m distance from the driving source with (W/L= 1.67). It is noticed that and confirmed by the previous analysis by getting far from the source of vibration the values of pore pressure start to be reduced by soil damping coefficient. For safety distance against vibration it is recommended to be in the range of 20 to 25 m away from the driving source. Sometimes there are no so many options to reach that safety distance therefore other methods for protection buildings is developed such as wave barriers.

Soil settlement is investigated at the same conditions for a building at 5m from the driving source. Settlement recorded a high values which are very hazard to the adjacent building and go beyond the limits set by the Egyptian code. It can be illustrated that it will not be accepted to process a sheet pile driving at 5m distance from the nearest building without the needed precautions. Differential settlement is very important indicator to the damage on the buildings, due to soil densification by the vibration and it is already known that the vibration amplitude decreased by the distance from the vibration source that will produce different values of soil densification along the distance from the vibration source which leads to differential settlement. Another effect of pile driving on the building is the distortion of the elements of the building itself, the building is vibrated directly due to the wave of vibration and will lead to increase in straining action of the structure elements. Displacement of the top of the building is measured by Plaxis and it shows different values in the both corners of the building.
That distortion of the buildings elements depends mostly on the wave length and the width of the building according to [10]. Figure 13 shows soil displacement and the building displacement at the top of it. It can be seen that soil displacement is high close to the building foundation which cause failure due to differential settlement beneath the foundation. The angular distortion of the top of the building in mm is $10/5.6 = 1.78$ and it obviously a great value for an 8m width building.

4.3. Effect of Different Sand Stiffnesses on Pore Pressure and Soil Settlement

Soil stiffness is very important parameter controlling the vibration amplitude because of the shape of particles and cohesion or friction between the soils particles, show how the waves of vibration propagate through the soil. Dense and hard soil produce high vibration amplitude compared with soft or loose soils according to [11]. Increasing in vibration amplitude leads to an increscent in excess pore pressure values. It can be seen that by increasing soil stiffness the pore pressure increase see Figure 14, which indicate that using driving piles in dense soil can increase the possibility of damage on adjacent buildings. Pore pressure is measured for three different soil stiffness at $E = 20000, 33000, 40000$ kN/m² at the foundation level (2m depth). It is noticed that the excess of pore pressure at soil stiffness of 33000 and
40000 kN/m² is relatively close and gives the same trend especially beyond about 20 m from the driving source where the values of Pore pressure start to decay. For pore pressure values at soil stiffness of E=20000 kN/m² it can be seen that the low values of pore pressure compared with the other cases of high soil stiffness. At 25 m and more the pore pressure values reach its minimum and there will be any possible hazard on buildings at that distance.

![Excess pore pressure for different soil stiffness](image1)

**Figure 14.** The excess of pore pressure due to driving sheet pile at different soil stiffness

On the other hand soil settlement increase in case of dense soil due to enlarge of densification by the great vibration amplitude, which leads to differential settlement. Figure 15 shows soil settlement values at different soil stiffness, it can be seen that at soil stiffness of 40000 kN/m² the soil settlement increase especially in the range 5 m from the driving source. Driving piles should not be processed at a distance less than 5 m.

![Soil settlement at different soil stiffness](image2)

**Figure 15.** The excess of pore pressure due to driving sheet pile at different soil stiffness
5. Conclusions

Effect of the pile driving on the excess of pore pressure and adjacent foundation response to born vibration are investigated using Finite Element Analysis. Based on the data and the results of the analysis presented in this paper, the following main conclusions can be drawn:

- Pore water pressure increased close to the pile toe and decreased by getting far from the vibration source.
- The ratio of D/L gives the optimum values and the safe criteria when it doesn’t exceeded 0.5.
- For the safe distance of buildings against the damage due to excess pore pressure is beyond 25m and with ratio W/L more than 2.
- Soil settlement reaches the highest values close to the driving source and the possible hazard increase in the range of 5m from the driving source.
- Increasing of soil stiffness increase the vibration amplitude which leads to an increase in pore pressure and soil settlement.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Effect of Vibrating Footing on a Nearby Static – Load Footing

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Abstract

This paper presents an experimental study on the dynamic response of square footings under effect of dynamic load comes from adjacent footing called the (source of vibration) which is excited by a known vibration source placed on the top of it, the objective is to study the effect of dynamic motion of the source of vibration on a nearby footing, called second footing, both footings rest on collapsible soil (gypseous soil) with gypseous content (60%). The study is performed through wide experimental program in dry and soaked condition. The first footing (source vibration) and the second footing have dimensions (80×80×40), (100×100×40) mm respectively and are manufactured from steel, then the two footings placed centrally over soil after prepared it in layers’ form in steel container with (1000×500×500) mm. The first footing exposed to vertical harmonic loading by using a rotating mass type mechanical oscillator to gives a similar effect of the dynamic loads, the second footing loaded with static weight only, under the dynamic excitation. The tests are conducted under dynamic response for three frequencies (10, 20, 30) Hz, the movement (displacement amplitude, velocity, and acceleration) of the second footing studied by varying spacing between the footings. The results showed that the amplitude of displacement, velocity, and acceleration for the second footing decreases when the spacing between footing increase. In addition, the value of these parameters at dry state is greater than its value at soaked state.

Keywords: Dynamic Load; Machine Foundation; Model Test; Square Footing; Vibration.

1. Introduction

The Dynamic loads can be generated from moving vehicles, heavy machines, or by move the train, etc. which causing the foundations to behave in a various mode under these loads. The problem of interaction between nearby foundations is of preponderant practical importance, as in many status foundations confront in fact are not secluded and they predominating interact with each other depending on of their close spacing, which may oftentimes be causing damage to structures in both strong and serviceability especially, under dynamic condition. Because of that, a need is to take out a simplified method to study the effect of foundations subjected to dynamic excitement on adjacent foundations. For single isolated foundation many researchers conducted a number of studies onset from the (simple spring-mass-dashpot) system to the (rigorous elastic half space model) proposed by many researchers like [1-3]. Analytical and numerical studies conducted to understand the dynamic interaction of foundations in a group and the soil-structure interaction behavior under dynamic loading such as [4-6].

The gypseous soil is one type of the collapsible soils; it covers wide areas in Iraq. This soil has high bearing capacity in dry state, but it subsides (collapses) upon saturation due to dissolving of cementation and particle bonding. Therefore, structures supported on unstable soil should be guarded against such danger. These problems are usually have led to
cracking, tilting and collapsing the related structure [7]. This type of soil covers about 31.7% in Iraq with different gypsum content ranging from 10-70% [8].

Chen (2015) researched the cross-cooperation issue among multi-establishments on a straight viscoelastic medium at little shear strains. In the examination, the establishments are discretized into various sub square-components. The dynamic reaction inside each sub-component is depicted by the Green's capacity. Consolidating the removal limit condition and the power balance of the establishments, it gets the dynamic impedance and consistence elements of the establishments. Broad outcomes for two unbending roundabout establishments put at various detachments are introduced. Parametric investigations are done on the dynamic collaboration among adjoining establishments and illustrative the outcomes for a few firmly dispersed establishments [9].

Abhijeet and Priyanka (2016) examined the dynamic cooperation impact of firmly separated square establishments under machine vibration through an exploratory examination, number of enormous scale model tests were directed in the field, the dynamic association of various blends of two-balance get together was researched by actuating vertical symphonious burden on one of the footings (dynamic balance), where the other balance (uninvolved balance) was stacked with the static weight as it were. The dynamic balance was energized with various sizes of dynamic stacking and the reaction was recorded for both the footings, set at various clear separating (S).

They were seen that the inactive balance experiences reverberation because of the dynamic excitation on the dynamic balance, which happens, notwithstanding, with a stage slack from the resounding recurrence of the dynamic balance. This stage slack is observed to be an indispensable parameter in characterizing the dynamic collaboration of a gathering of footings. The stage slack is seen to diminish with abatement in the dispersing between the footings, the variety of transmission proportion is by and large connected with a base and a most extreme point [10].

Chen (2016) performed parametric investigation on the dynamic cooperation between neighbouring establishments. The impacts of separation and arrangement course between establishments have been analysed. He detailed that the consequences of establishments adjusted along various heading showed that the impedance capacities because of the comparing loads on the establishment itself, change pretty much nothing. The coupling impedance works because of the relating loads on different establishments, are emphatically influenced by the establishment arrangement heading. The impedance of an establishment in an establishment bunch at bigger recurrence and separation will in general be that of single establishment [11].

Sbartai (2016) directed an examination on the dynamic cooperation for two neighboring unbending establishments implanted in a viscoelastic soil layer. The technique which has been utilized is the limit component strategy (BEM) to define the arrangement, at that point to deciding the consistence elements of the two adjoining establishments concerning their dispersing, substratum profundity, masses, shapes, installing, load force, and frequencies of excitation. The examination of the exhibited investigation showed that the impact of a few parameters on the dynamic communication reaction of two nearby establishments is no immaterial. Specifically, the predominant impact of certain parameters, for example, the heterogeneity of the dirt, state of the establishments, and the heap force, contrasted with different ones is plainly uncovered [12].

Keawsawasvong and Teerapong (2017), researched the dynamic communication between two inflexible rectangular establishments and a multi-layered poroelastic medium exposed to time-symphonious vertical stacking, it accepted the contact surface between the establishment and the layered medium to be smooth and completely penetrable, and the contact surface was discretized into various square components. It is discovered that the impedance capacities are obviously impacted by the recurrence of excitation and the separation between the two establishments [13].

Han et al. (2017) explored the dynamic association between at least two adjoining establishments laying on the outside of a stratified soil. Parametric investigation was done to explain the impacts of layer profundity, soil damping, dividing between adjoining establishments, masses and minute in activities of supporting structures and the wave engendering speed on the dynamic conduct of three-dimensional (3D) establishment soil–establishment communication (FSFI). Likewise, numerical precedents are given to confirm the exactness and computational dependable of the proposed methodology. He referenced the interfaces between the establishments and the dirt should be discretized and there is no restriction on the thickness or on the quantity of soil layers to be considered and study demonstrated the numerical outcomes got are exceptionally precise [14].

Ali et al. (2018) displayed the aftereffects of the dynamic investigation of a four-chamber blower establishment. The establishment square backings a four-chamber dress-rand blower, suction and release bottles. What's more, by utilizing a three-dimensional limited component model of the dirt establishmen framework to decide the dynamic reaction of the dirt establishment framework and to survey the establishment reaction under the connected powerful stacking forced by the blower wrench. The dynamic examination is performed by (1) performing eigenvalue investigation of the establishment square, considering the impact of the dirt establishment connection to decide the dirt establishment characteristic frequencies and modular support variables, and (2) performing constrained reaction of the establishment
under connected crankshaft unbalance burden to decide the constrained reaction adequacy of the dirt establishment framework [15].

Vicencio and Nicholas (2018) assessed the impact of Structure-Soil-Structure Interaction (SSSI) between two structures given various parameters of the structures, between structure dispersing, and soil type. He proposed a two-dimensional straightforward discrete nonlinear model and depicted this by a lot of nonlinear differential conditions of movement. The outcomes demonstrated that there are both horrible and helpful setups of the two structures that produce significant contrasts between nonlinear SSSI and nonlinear SSI (the uncoupled structure case). He referenced that the unfavourable impacts of SSSI can be increasingly articulated when the nonlinear soil conduct is accepted [16].

Andersen (2018) examined the significance of dynamic structure–soil–structure association (SSSI) for structures with at least two establishments, it led the investigation of such polypod establishments in recurrence space, thinking about the range (0 – 50) Hz and utilizing Green's function for wave engendering in layered soil. The standardized powerful solidness identified with individual establishments and cross-coupling between two establishments are introduced [17].

Keawsawasvong et al. (2019), exhibited an examination on the dynamic association issue including firmly dispersed establishments under shaking vibrations. It is discovered that shaking vibrations of the stacked establishment could be considered as a solitary establishment when the separation between the contiguous establishments is twice more noteworthy than their width. Despite what might be expected, an emptied establishment would in any case experience a transmitted vitality disseminated from a stacked establishment notwithstanding when the separation between them increments [18].

Therefore, lack of experimental studies on the behavior of foundation which are adjacent to the machine foundations encourage to take up the present’s investigation, that gives an explanation of the dynamic interference effect between two closely spaced square footings resting on gypseous soil by conducting small scale model experimental.

The development in the cities as a result of the urgent need to use machines and equipment, and these devices are considered as main sources of vibrations which transfer through soil and effect on their engineering properties. So the objectives of the current work are investigating experimentally the dynamic response of foundation resting on a gypseous sandy soil under the effect of dynamic load installed on the adjacent foundation in both state (dry and soaked). The dynamic response includes displacement amplitude, velocity of vibration, and the acceleration.

This paper presents a study on dynamic response of one footing with static load nearby another footing with dynamic load as (source of vibration) and dynamic loading applied on top of it, both footing have square shape, on a gypseous soil. The present’s investigation deals with the effect of the vibrations of the first footing on the neighboring footing. Both foundations are resting on surface soil.

1.1. Definition of Problem

Two closely spaced square placed on gypseous soil (in both state dry and soaked) as shown in Figure 1. The load intensity below the first footings (source of vibration) was maintained as (6 kN/m²), the second footing is placed with intensity of load equal to (30 kN/m²) and (S) represents the spacing between the two footings. The objective is to determine the dynamic response of the second footing (displacement amplitude, velocity, acceleration), due to the application of dynamic excitation on the top of the first footing.

![Figure 1. Layout of experimental model](image)
2. Materials and Methods

2.1. Apparatuses of Model

The apparatuses of model include the followings:

1) Steel box with dimension (1000x500x300) mm.
2) Two footings with dimensions (80x80x40) mm, for first footing as a machine foundation, (100x100x40) mm, for second footing which manufactured from steel both footings have square shape.
3) Mechanical oscillator.
4) Piezoelectric accelerometer.
5) Two dial gauge.
6) Variable frequency drive.
7) Vibration meter.
8) Digital tachometer.
9) Computer device.
10) Steel Mold.
11) Water tank.
12) Dial gauge.
13) Static weight.
14) Ac automatic voltage regulator.
15) Camera stand

2.2. Test Setup

After reviewing the previous studies presented by various researchers, such as [19 -24], the experimental model setup was designed, on the analysis of foundations under effect of the harmonic vibrations. A two steel square footings, the first with dimensions (100x100x40) mm as a machine foundation (vibration source), the second footing with dimensions (80x80x40) mm subjected to static load only. Circular weights (20kg in mass) of 25cm in diameter used for loading the second footing statically, a rotating mass mechanical oscillator instead over the first footing to produce a varying dynamic load. The mechanical oscillator be composed of a rotating discs made from steel, with diameter (60) mm and, thickness (10) mm. A small eccentricity mass (me) is put on rotating disc at an eccentricity (e) of (20) mm, from the axis of rotation. In this study, one type of eccentric settings is used with value (60) gm.

A varying speed DC motor is used to turn on the (mechanical oscillator) at different frequency with ranging from (100rpm – 11000) rpm. A speed controller unit is placed outside the model to control the speed of the DC motor.

The Piezoelectric accelerometer which was connected directly to computerized digital vibration meter (6063) model, which was in turn interfaced to the computer for displaying the dynamic response as displacement amplitude, velocity or acceleration is set before the test.

The digital tachometer (DT-2234A+) model was used to ensure that there is no change in the frequencies. Two-dial gauge are used to determine the settlement of the second footing during operation model by Place them on the edges of the footing.
2.3. Preparation and Test Procedure

The soil in this research was taken from Governorate located north of Iraq, namely Tikrit, has been carried out for the testing program; see Figure 5. Table 1 shows the physical properties of the soil and Table 2 shows the chemical properties. Figures 6 and 7 revealed the results of the laboratory tests carried out on the soil sample used in current study. Water content test is conduct at temperature (45 °C) to prevent losing crystal of gypseous soil. The sample of gypseous soil of gypseous content of (60%) are classified as moderately severe (ASTM D5533-2003). The gypseous soil (passing sieve no.4) placed in the steel container in six layers with a uniform field density using the hummer device. The surface checked and leveled by a bubble ruler (balance) device. Both footings are placed centrally over the prepared soil.
Table 1. Physical property of gypseous soil which is used for testing

<table>
<thead>
<tr>
<th>Test</th>
<th>Properties</th>
<th>Value</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Specific Gravity (Gs)</td>
<td>2.41</td>
<td>ASTM D 854 (2006)</td>
</tr>
<tr>
<td>Atterberg’s limits</td>
<td>Liquid limit (L.L) %</td>
<td>21.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Plastic limit (P.L) %</td>
<td>N.P</td>
<td>ASTM D4316-84</td>
</tr>
<tr>
<td></td>
<td>Plasticity Index (P.I)</td>
<td>N.P</td>
<td></td>
</tr>
<tr>
<td>Compaction characteristics</td>
<td>Max. dry density (KN/m³)</td>
<td>16.23</td>
<td>ASTM 698-00</td>
</tr>
<tr>
<td></td>
<td>Optimum Moisture content %</td>
<td>12.33</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Water content %</td>
<td>2.8</td>
<td>ASTM D2216-02</td>
</tr>
<tr>
<td>Grain size analysis</td>
<td>D10 (mm)</td>
<td>0.07</td>
<td>ASTM D422-02</td>
</tr>
<tr>
<td></td>
<td>D30 (mm)</td>
<td>0.14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D60 (mm)</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Coefficient of uniformity, Cu</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Coefficient of curvature, Cc</td>
<td>0.8</td>
<td></td>
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<tr>
<td></td>
<td>Passing sieve No. 200 (%) (using kerosene)</td>
<td>24</td>
<td></td>
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<tr>
<td></td>
<td>Classification of soil based on (USCS)</td>
<td>SM</td>
<td></td>
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<tr>
<td>The collapse potential</td>
<td></td>
<td>7.9</td>
<td>ASTM D5533-2003</td>
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<tr>
<td>Direct Shear Test</td>
<td>Angle of Internal Friction (Ø) in dry</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soil Cohesion (C) (KN/mm²) in dry</td>
<td>14</td>
<td>ASTM D 3080-98</td>
</tr>
<tr>
<td></td>
<td>Angle of Internal Friction (Ø) in soaked</td>
<td>34</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soil Cohesion (C) (KN/mm²) in soaked</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Test unit weight (kN/m3), γd test</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Field density ((kN/m³), γfield</td>
<td>14.6</td>
<td>ASTM D1556-07</td>
</tr>
</tbody>
</table>

Table 2. Results of chemical properties of gypseous soil used for testing (BS 1377: 1990, Part 3)

<table>
<thead>
<tr>
<th>Composition</th>
<th>Value %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total soluble salts (T.S.S.)</td>
<td>67.2</td>
</tr>
<tr>
<td>Gypsum content %</td>
<td>60</td>
</tr>
<tr>
<td>Sulphate content (SO3) %</td>
<td>30.5</td>
</tr>
<tr>
<td>Organic matters (O.M) %</td>
<td>0.22</td>
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<tr>
<td>Chloride content (CL) %</td>
<td>0.062</td>
</tr>
<tr>
<td>pH value</td>
<td>8.1</td>
</tr>
</tbody>
</table>

Proper care is taken to keep the center of gravity of system and the footings to lie in the same vertical line with center of gravity of the container. After studying results reached by the researchers and carried out some preliminary tests, time of one hour for the duration of the tests chosen. 30 minutes’ operation test for the dry state and 30 minutes for the test under soaking condition, it is important to mention that for the test under soaking condition, steel container left for (24) hours to be sure that soil was completely soaked, and in the second day the test is continue.

In this investigation, eccentric settings (me = 60 gm) is used to simulate dynamic load. The oscillator is then operated slowly through a motor by using speed control unit to prevent sudden application of high dynamic load. Thus, the first footing subjected to vibration in the vertical direction.

The dynamic response (displacement amplitude, velocity, acceleration) of the second footing are measured and recorded at the same time using Piezoelectric accelerometer and two dial gauge. are placed, previously on top and edge of the second footing to obtain a foundation response, the operating frequency of (600, 1200, 1800) rpm equivalent to (10, 20, 30) Hz is considered in the present study, the response parameters were recorded every 5 minutes during operation test duration.
Figure 8. Laboratory Model Testing

3. Results and Discussion

In the present study, the dynamic response of square footing is investigated under dynamic exciting force which originated from a nearby footing as a machine foundation.

After verifying the stability of foundations under the static load, the dynamic analysis for the second footing is carried out by exciting the first footing with vertical load intensity created by the machine vibration. The displacement amplitude, velocity, and acceleration at different spacing between the two footings (S=B, S=2B, S=3B) are measured which the two footings are erected on gypseous soil for both state (dry and soaking) in presence of the dynamic excitation applied on the first foundation. The dynamic response of the second footing for three frequencies (10, 20, 30) Hz at different spacing between footings is shown in Figures 9-14.

3.1. Displacement Amplitude

In Figures 9 and 10 the maximum and minimum displacement amplitude plotted against the frequency, for three frequencies (as mentioned above) at dry and soaking state, we note that the magnitude of the displacement amplitude at (S = B) increases when the operation frequency increases for both state (dry and soaking). For the maximum amplitude at dry and soaking state, this increasing is slight when goes from 10 Hz to 20 Hz and be larger when goes to 30 Hz. On other hand, the minimum amplitude shows increases just slightly when goes form 10 Hz to 20 Hz or 30 Hz at dry state but at soaking, its doubled in value.

At soaking state, it is observed that there is a decreasing in magnitude of the displacement amplitude compare with dry state for the three frequencies, this decreasing after soaking can be attributed to presence of water, whom acting as a wave inhibitor. The energy of the vibrations generated from the first footing became lower during transmission through soil at soaking condition causing decreasing in the displacement amplitude of the second footing.

At (S=2B), here finds decreasing in values of the maximum and minimum displacement amplitude compare with magnitude of the displacement amplitude at (S=B) for both state (dry and soaking). This is because the wave of vibration cut off a longer distance when it travels from the source (the first footing) to the receiver (the second footing). In other word, increasing the spacing led to decreases the displacement amplitude, the propagated of vibrations through the soil leads to a decrease in the energy of those vibrations.

At (S=3B), for the maximum displacement amplitude at dry state, the magnitude increased by half when goes from 10 Hz to 20 Hz and increased by two and a half times in value at 30 Hz. For the minimum displacement amplitude, the value has doubled from 10 Hz to 20 Hz or 30 Hz.
At soaking state, there is a decreasing in magnitude of displacement amplitude compare with dry state. The maximum displacement amplitude shows slight increase when goes from 10 Hz to 20 Hz, but doubled in magnitude when moves to 30 Hz. The minimum displacement amplitude shows different behavior, the magnitude of amplitude increases doubled when goes from 10 Hz to 20 Hz and increased by three times in value at 30 Hz.

Here observes the magnitude of the displacement amplitude decreased in comparison with magnitude amplitude at (S=B), and (S=2B) for both state (dry and soaked).
As a result, it can mention the general remarks as followings:

- The displacement amplitude at soaked state less than the displacement amplitude at dry state for all frequencies.
- The displacement amplitude, increase when frequency increase, regardless whether the state is wet or dry.
- As a logic, the maximum displacement amplitude reduced when the spacing between the two footings increase, in both soaked and dry specimen.
3.2. Velocity of Vibrations

Figures 11 and 12 show the relationship between the maximum and minimum velocity versus the frequency which are recorded for three spacing (S=B, S=2B, S=3B) on dry and soaked soil basis, it can be seen that maximum and minimum velocity diverge with increases of frequency, and converge with increased of spacing between the two footings for both state (dry and soaked).

The value of the velocity of the second footing increases with the increase the frequency of the first footing, on the other hand, the magnitude of the velocity decreased with increasing the spacing between the two footings, that’s because the propagated of vibrations through the soil leads to a decrease in the energy of those vibrations and therefor decrease

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**Figure 11.** The velocity versus frequency for different spacing (s), (a) at S=B, (b) at S=2B, (c) at S=3B for dry condition

The value of the velocity of the second footing increases with the increase the frequency of the first footing, on the other hand, the magnitude of the velocity decreased with increasing the spacing between the two footings, that’s because the propagated of vibrations through the soil leads to a decrease in the energy of those vibrations and therefor decrease
the velocity of vibrations. In addition, the value of velocity at dry state is greater than its value at soaked state, because of the presence of water in soil (whom acting as a wave inhibitor), causing decreasing in energy of vibration and that led to decrease in velocity of vibration for the second footing.

As a result, it can mention the general remarks as followings:

- The velocity of vibrations at dry state greater than the velocity of vibrations at soaking state for all frequencies.
- The velocity of vibrations, increase when frequency increase, regardless whether the state is wet or dry.
- The velocity of vibrations reduced when the spacing between the two footings increase, in both soaked and dry specimen.

![Figure 12. The velocity versus frequency for different spacing (s), (a) at S=B, (b) at S=2B, (c) at S=3B for soaked condition]

### 3.3. The Acceleration

The maximum and minimum acceleration versus the frequency are shown in Figures 13 and 14, which are recorded for three spacing of (B) (S=B, S=2B, S=3B) for dry and soaked condition. The trend of behavior for maximum and
minimum acceleration is similar in dry and soaked condition, it can be seen the value of the acceleration increases with increase the frequency in both state (dry and soaking) and decreases with increase the spacing between the footings. The effects of increasing the spacing between the footings on the magnitude of acceleration are similar to the effect on displacement amplitude and velocity, increasing the spacing led to decreases the acceleration, that’s because the propagated of vibrations through the soil leads to a decrease in the energy of those vibrations (as mention earlier) and therefore decrease the acceleration. In addition, we observe the magnitude of the acceleration at dry state is greater than its value at soaked state.

At dry state, the maximum and minimum acceleration shows increases in magnitude of the acceleration almost linearly when goes from 10 Hz to 20 Hz or 30 Hz, at spacing (1B). The same goes when spacing increase to (2B) and (3B). And here observed, the gap between the maximum and minimum increases when the frequency increase and the gap decreases when the spacing between the two footings increases. See Figure 13.

![Graph](image1)

**Figure 13.** The Acceleration versus frequency for different spacing (s), (a) at S=B, (b) at S=2B, (c) at S=3B for dry condition
At soaking state, for the maximum and minimum acceleration, the rate of increase in acceleration just slightly when goes form 10 Hz to 20 Hz, but for 30 Hz is about higher than a double value, this applies to the magnitude of acceleration at a spacing of 1B, 2B, and 3B. The gap between the maximum and minimum acceleration increases when increasing the frequency and decreases with increases the spacing between the two footings. The values of the acceleration at soaked state for three frequencies (10, 20, 30) Hz at spacing of 1B, 2B, and 3B, recorded decreasing compared with their values at dry state because of the presence of water in soil whom acting as a wave inhibitor (as mentioned earlier), and it, increase when frequency increase, regardless whether the state is wet or dry. The acceleration reduced when the spacing between the two footings increase, in both soaked and dry specimen, see Figure 14.

Figure 14. The Acceleration versus frequency for different spacing (s), (a) at S=1B, (b) at S=2B, (c) at S=3B for soaking condition
4. Conclusions

4.1. Displacement Amplitude

- The magnitude of displacement amplitude of the foundation under effect of dynamic load comes from adjacent foundation (both foundations erected on gypseous soil) increases with increase the operation frequency.
- The value of amplitude at dry state is greater than its value at soaked state.
- The displacement amplitude decrease with increases the spacing between the two footings. The reduction in value of displacement amplitude when the spacing between the two footings increased from 1B to 2B at frequency of 10Hz is 15.3 % at dry state, and 21% at soaking state. and when the spacing increased from 1B to 3B, the reduction is 44.6% and 52.9% at dry and soaking state respectively, at frequency of 20 Hz, the displacement amplitude decreased by (9.35% and 13.8%) at dry and soaking state respectively, when spacing increased from 1B to 2B and decreased by (13.3% and 22.4%) at dry and soaking state respectively, when spacing increased from 1B to 3B. For the frequency of 30 Hz, the reduction is 15.7% at dry state and 36.5% at soaking state when the spacing increased from 1B to 2B, and when the spacing increased from 1B to 3B, the displacement amplitude decreased by (19% and 41.7%) at dry and soaking condition respectively.

4.2. Velocity of Vibrations

- The velocity of vibrations of foundation (loaded with static weight) nearby another foundation which subjected to dynamic load (both foundations erected on gypseous soil) increases with the augments of operation frequency.
- The value of the velocity decreases with the increase of spacing between the two footings. The reduction in value of velocity of vibrations when the spacing between the two footings increased from 1B to 2B at frequency of 10 Hz is 11.6 % at dry state, and 38% at soaking state. and when the spacing increased from 1B to 3B, the reduction is 43.8% and 59.8% at dry and soaking state respectively, at frequency of 20 Hz, the velocity of vibrations decreased by (21.3% and 45%) at dry and soaking state respectively, when spacing increased from 1B to 2B and decreased by (43% and 48.7%) at dry and soaking state respectively, when spacing increased from 1B to 3B. For Frequency of 30 Hz, the reduction is 2% at dry state and 1% at soaking state when the spacing increased from 1B to 2B, and when the spacing increased from 1B to 3B, the velocity of vibrations decreased by (10% and 11.5%) at dry and soaking condition respectively.

4.3. Acceleration

- The acceleration of the foundation under effect of dynamic load comes from adjacent foundation on gypseous soil decreases with decreases the operation frequency.
- The value of the acceleration at soaking state is lower than its value at dry state.
- The value of the acceleration decreases with the increase of spacing between the two footings. The reduction in magnitude of acceleration when the spacing between the two footings increased from 1B to 2B at frequency of 10 Hz is 24 % at dry state, and 30% at soaking state. and when the spacing increased from 1B to 3B, the reduction is 51% and 59% at dry and soaking state respectively, at frequency of 20 Hz, the acceleration decreased by (22.2% and 18%) at dry and soaking state respectively, when spacing increased from 1B to 2B, and decreased by (45.1% and 54.5%) at dry and soaking state respectively, when spacing increased from 1B to 3B. For Frequency of 30 Hz, the reduction is 29.7% at dry state and 24.2% at soaking state when the spacing increased from 1B to 2B, and when the spacing increased from 1B to 3B, the acceleration decreased by (32.4% and 27.3%) at dry and soaking condition respectively.

5. Acknowledgement

I would like to express deep gratitude to my supervisor Assist. Prof. Dr. Waad A. Zakaria for encouragement and significant suggestions, which improved this work. So, I’m greatly indebted to him. Appreciation and thanks are also extended to the all staff of Civil engineering department, and the staff of Soil Mechanics Laboratory, Diyala University, Iraq.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Spatiotemporal Dynamics of Land Surface Temperature and Its Impact on the Vegetation

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Abstract

Due to global warming under climate change scenarios, Indus delta region of Pakistan is under serious threat since the last few decades. The present study was thus conducted to determine the spatiotemporal variations in the LST and its impact on the vegetation of the Indus delta, using satellite data for the past 27 years (1990-2017). The analysis revealed that on average, there was an increase of 1.74 °C in LST during the last 27 years. The temporal variation in the Normalized Difference Vegetation Index (NDVI), an indicator of vegetation, showed the highest NDVI of 0.725 in the year 2005 followed by the year 2010 with NDVI of 0.712. While the lowest NDVI of 0.545 was observed during the year 2017. The LST was integrated with NDVI which showed a fair but negative statistical correlation with a coefficient of determination R² = 0.65. A correlation analysis between NDVI and the yield of the wheat crop of the Delta showed a positive relationship with R² = 0.89. Several factors may contribute to an increase in LST, such as an increase in residential areas, change in the cropping pattern and overall global climate change. Such studies are important for determining the climatic influences on ecological parameters.

Keywords: LST; NDVI; Crop Yield; Spatiotemporal Analysis; Coastal Areas.

1. Introduction

Due to the global climate change issues, the land surface temperature (LST) has increased, which has affected land use, land cover, vegetated areas, water resources, etc. Chan and Yao [1], and Choudhury et al. [2] reported that such changes are responsible for various environmental problems. LST refers to the temperature of the earth surface including the temperature of bare soil, the canopy of vegetation, etc. [3-4]. For hydrologists, agronomists, amenagists, meteorologists, the information of different terms, which interfere with the energy balance of the surface is very important. However, the LST is one of the key parameters which plays a vital role in the processes of interaction between hydrosphere, biosphere, and atmosphere. The LST is also used in many fields such as climate change, evapotranspiration, hydrological cycle, vegetation, etc. [5]. It is the main parameter which is affected by the properties of land surface such as land use, land cover, vegetation, and type of vegetation as well as the permeability of the surface of the soil [3]. Numerous studies have been conducted to observe the changes in the LST in the result of variations in land use land cover, vegetation. Most of the studies have reported an inverse relation between LST and vegetative cover, which refers that crop cover decreases the LST.

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At present, many researchers use the GIS and Remote Sensing techniques to explore the relation between LST, NDVI and land use land cover [6]. Dagliyar et al. [7] described that thermal bands of satellite imagery can be used to determine the LST. They determined the LST in the Erzurum, Turkey, using the OLI/TIRS data acquired on July 25, 2014. Various types of vegetation indices are available which can be used to observe changes in the vegetative area. But NDVI is one of the efficient, simplest and widely used index [8]. Using this index, the changes occurred in the vegetation for any specific area can be quantified. Two types of approaches such as the conventional approach and the approach of Remote Sensing (RS) can be used to quantify the LST. In the conventional approach, the LST is determined from weather stations, while by RS make it possible to estimate it through the model of the energy balance of surface [9]. Rajendran and Mani [10] stated that LST is a significant variable of microclimate and transfer of radiation within the atmosphere. GIS and RS tools integrated with ground truthing field data are recommended to assess the spatiotemporal variations in the LST. In a study conducted on LST in Thiruvananthapuram, city of India using the thermal bands of Landsat 8, OLI/TIRS imagery revealed that LST is a function of water content and vegetative cover of soil. Yue et al. [11] used satellite data to determine the relationships between the LST and NDVI in the Shanghai city of China and reported GIS and RS tools as effective in determining the climatic influences on the ecosystem. Fast changes in patterns of land use and land cover have brought significant changes in LST.

Similar is the case of the Indus delta, Sindh, Pakistan, where, increasing LST has significantly affected the vegetative cover of the region [12]. Rehman et al. [13] also reported that the LST in the Keti Bandar area of Sindh province of Pakistan is increasing, and adversely affecting the vegetation of the area. It has been reported that due to global warming and climatic change, coastal areas of Sindh, Pakistan are under serious threat since the past few years. Keeping in view and gravity of the problem, the present study was carried out to determine the spatial and temporal variation in LST of the Indus Delta, Sindh, Pakistan using multispectral satellite data for the period of the last 27 years (1990-2017). The extracted LST data were integrated with NDVI to see the relationship between these parameters. Also, NDVI of the delta was correlated with wheat crop yield using the Pearson correlation model. The findings of the study will be useful for environmentalists, policymakers, farmers of the area for taking remedial measures to mitigate climate change impacts in the region.

2. Study Area

The Indus River makes its delta when it empties into the Arabian Sea. Most of its area stretches in two district administrative boundaries of Sindh Province, Pakistan, i.e., Thatta, and Sujawal (Figure 1). The area hardly receives about 220 mm of rainfall annually [14-18], mostly falls during monsoon periods. According to the 2017 Census, about 1.76 million people are living in these two districts. In the past, the delta was counted as one the prosperous areas of Indus civilization, but now it is counted as poorest areas of Pakistan. Due to climate change scenarios, the delta is shrinking and degrading at an alarming rate [19-20]. Reduction of freshwater flows and entry of nutrient-rich sediments into the Indus River and resulting seawater intrusion into its delta have adversely affected the vegetative cover, flora, fauna, water resources, soil fertility, as well as socioeconomic conditions of the community.

3. Materials and Methods

3.1. Quantification of Land Surface Temperature (LST)

To determine the relation between LST and vegetation of the Indus delta, thermal bands of satellite imageries for the last 27 years (1990-2017) as described in Table 1 were used. The satellite data of the TM (thematic mapper), ETM+.
(enhanced thematic mapper plus), and OLI/TIRS (operational land imagery/thermal infrared sensor) sensors for the
Earth Explorer website. The digital numbers (DNs) of the thermal bands (band 6 for Landsat 5 and 7 and band 10 for
Landsat 8) of the AOI (area of interest) were first converted to top of atmospheric spectral radiance (ToA,) and then to
LST using following two steps.

3.2. TOA Radiance Conversion

Some thermal, electromagnetic energy is reflected by every object when it is placed in temperature $>0$, which is
known as absolute zero [2]. According to this principle, the thermal sensors receive signals, which can be converted into
at-sensor radiance. In the present study, the spectral radiance ($L_a$) was calculated using Equation 1.

$$ L_a = M_L Q_{cal} + A_L $$

(1)

Where, $L_a$ - Top of Atmosphere Spectral Radiance (TOA); $M_L$ - Band-specific multiplicative rescaling factor obtained from the metadata file; $A_L$ - Band-specific additive rescaling factor obtained from the metadata file; $Q_{cal}$ - Quantized and calibrated standard product pixel values (DNs).

3.1.2. Conversion of Spectral Radiance ($L_a$) into Brightness Temperatures (LST)

Then thermal band data were converted from TOA, to land surface temperature (LST). As the LST data were obtained
in the Kelvin scale, which is not common now, hence, it was converted to centigrade scale by subtracting 273 from
every pixel, using ArcGIS 10.5 software, as explained in Equation 2.

$$ T = \frac{K_2}{\ln \left( \frac{K_1}{L_a} + 1 \right)} - 273 $$

(2)

Where; $T$ - At-satellite brightness temperature (K); $L_a$ - TOA spectral radiance; $K_1$ and $K_2$ - Band-specific thermal conversion constants from the metadata;

The satellite images used in the study along with values of band-specific multiplicative rescaling factor ($M_L$), band-
specific additive rescaling factor ($A_L$), and thermal conversion constants ($K_1$ and $K_2$) are summarized in Table 1.

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<th>Band Number</th>
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<th>$M_L$</th>
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<td>B-6</td>
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<td>1260.56</td>
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<td>Feb. 14, 2000</td>
<td>43</td>
<td>151</td>
<td>B-6</td>
<td>-0.06709</td>
<td>0.067</td>
<td>666.09</td>
<td>1282.71</td>
</tr>
<tr>
<td>Feb. 05, 2000</td>
<td>43</td>
<td>152</td>
<td>B-6</td>
<td>-0.06709</td>
<td>0.067</td>
<td>666.09</td>
<td>1282.71</td>
</tr>
<tr>
<td>Feb. 27, 2005</td>
<td>43</td>
<td>151</td>
<td>B-6</td>
<td>-0.06709</td>
<td>0.067</td>
<td>666.09</td>
<td>1282.71</td>
</tr>
<tr>
<td>Feb. 18, 2005</td>
<td>43</td>
<td>152</td>
<td>B-6</td>
<td>-0.06709</td>
<td>0.067</td>
<td>666.09</td>
<td>1282.71</td>
</tr>
<tr>
<td>Feb. 09, 2010</td>
<td>43</td>
<td>151</td>
<td>B-6</td>
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<td>0.067</td>
<td>666.09</td>
<td>1282.71</td>
</tr>
<tr>
<td>Feb. 16, 2010</td>
<td>43</td>
<td>152</td>
<td>B-6</td>
<td>-0.06709</td>
<td>0.067</td>
<td>666.09</td>
<td>1282.71</td>
</tr>
<tr>
<td>Feb. 15, 2015</td>
<td>43</td>
<td>151</td>
<td>B-10</td>
<td>0.10000</td>
<td>0.000334</td>
<td>774.89</td>
<td>1321.08</td>
</tr>
<tr>
<td>Feb. 06, 2015</td>
<td>43</td>
<td>152</td>
<td>B-10</td>
<td>0.10000</td>
<td>0.000334</td>
<td>774.89</td>
<td>1321.08</td>
</tr>
<tr>
<td>Feb. 20, 2017</td>
<td>43</td>
<td>151</td>
<td>B-10</td>
<td>0.10000</td>
<td>0.000334</td>
<td>774.89</td>
<td>1321.08</td>
</tr>
<tr>
<td>Feb. 11, 2017</td>
<td>43</td>
<td>152</td>
<td>B-10</td>
<td>0.10000</td>
<td>0.000334</td>
<td>774.89</td>
<td>1321.08</td>
</tr>
</tbody>
</table>

1Landsat 5; 2Landsat 7; 3Landsat 8.

3.2. Method for Calculation of Normalized Difference Vegetation Index (NDVI)

The relation between LST and vegetation of the Indus delta in terms of a spectral index NDVI (normalized difference
vegetation index) was calculated using satellite data and ArcGIS 10.5 software. This index was calculated from the NIR
This index has been widely used by various researchers around the globe as an indicator of greenness. For Landsat 5 and 7, bands 3 and 4, however, for Landsat 8/OLI, bands 4 and 5 were used to calculate NDVI. The index values vary from -1 to +1. A value of -1 characterizes a non-vegetated area, while +1 represents the vegetative area.

\[
\text{NDVI} = \frac{\text{Near Infrared} - \text{Red}}{\text{Near Infrared} + \text{Red}}
\]  

(3)

3.3. Effect of LST on the Vegetation of the Delta

To see the effect of LST on the plants/vegetation of the Indus delta, the LST was integrated with NDVI. Regression analysis was performed between these two parameters to show how the changing LST create an effect on the vegetative cover of the Indus delta, as explained by Choudhury et al. [2].

3.4. The Relation between NDVI and Crop Yield of the Delta

To see the relation between NDVI and wheat crop yield of the Indus delta (collected from Sindh Agricultural Extension Department), a regression analysis was performed between these two parameters. Figure 2 shows the flowchart of the methodology adopted in this study for determination of LST and its impact on the vegetation.

---

Figure 2. Flowchart of the methodology

---

4. Results and Discussions

4.1. Land Surface Temperature

The spatial and temporal variation in LST of the Indus delta during 1990-2017 is portrayed in Figure 3. It shows that areas with water and vegetation have lower LST compared to towns and barren land. Temporal variation in the Indus delta under different temperature ranges is quantified in Table 2. The temporal change in LST is more prominent in the south-east of the delta (near to Sir Creek). It may be because of change in hydrological features of the area due to the construction of Tidal Link Canal in the late 1990s. The area under a temperature above 30°C increased temporally from 511 ha in 1990 to 115,304 ha in 2017 as shown in Figure 4, which confirms the temporal increase in LST in the delta as also reported by Rehman et al. [13]. Thus, the area of the delta having LST above 30 °C has increased more than 225 times in the last 27 years. Several factors may contribute to a temporal increase in LST, such as an increase in residential areas and overall global climate change.
Figure 3. Spatial and temporal variation in the LST (1990-2017)
Table 2. The variation in the area under different land surface temperature ranges

<table>
<thead>
<tr>
<th>Year</th>
<th>&gt;15 °C</th>
<th>15-20 °C</th>
<th>20-25 °C</th>
<th>25-30 °C</th>
<th>&gt;30 °C</th>
<th>Total (ha)</th>
<th>%</th>
<th>Total (%)</th>
<th>Average LST</th>
</tr>
</thead>
<tbody>
<tr>
<td>1990</td>
<td>570</td>
<td>319627.2</td>
<td>771611</td>
<td>214390.4</td>
<td>511</td>
<td>1306710</td>
<td>100</td>
<td>100</td>
<td>22.10</td>
</tr>
<tr>
<td>1995</td>
<td>0</td>
<td>533633.6</td>
<td>621326</td>
<td>150589</td>
<td>1168</td>
<td>1306716.6</td>
<td>100</td>
<td>100</td>
<td>21.08</td>
</tr>
<tr>
<td>2000</td>
<td>322</td>
<td>284694</td>
<td>695205</td>
<td>305309</td>
<td>18281</td>
<td>1306717</td>
<td>100</td>
<td>100</td>
<td>22.70</td>
</tr>
<tr>
<td>2005</td>
<td>123948</td>
<td>601129</td>
<td>455857</td>
<td>31873</td>
<td>1298291</td>
<td>100</td>
<td>20.04</td>
<td>20.04</td>
<td></td>
</tr>
<tr>
<td>2010</td>
<td>3.96</td>
<td>254956</td>
<td>459033</td>
<td>523691</td>
<td>58343</td>
<td>1296027</td>
<td>100</td>
<td>23.99</td>
<td></td>
</tr>
<tr>
<td>2017</td>
<td>52</td>
<td>272896</td>
<td>525128</td>
<td>393385</td>
<td>115304</td>
<td>1306765</td>
<td>100</td>
<td>23.84</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4. Dynamics the study area having LST >30 °C during a period of last 27 years

On average, there was an increase of 1.74 °C in LST in the Indus delta during the period of the last 27 years as described in Table 2.

4.2. Temporal Change in the NDVI of the Indus Delta

The temporal variation in the NDVI of the Indus delta is presented in Figure 5. It shows that the highest NDVI of 0.725 was in the year 2005 followed by the year 2010 with NDVI of 0.712. While the lowest NDVI of 0.545 is for the year 2017. These NDVI values are reflected in Figure 5, which shows the temporal change in the area under vegetation of the delta. The green color in Figure depicts the positive NDVI values with lush green vegetation while brown color represents water and barren land with negative values of the NDVI.
4.3. Effect of LST on the Vegetation of the Delta

The impact of LST on the vegetation of the delta was evaluated by correlating LST and NDVI as shown in Figure 6. It shows that there was a fair but negative statistical correlation between NDVI and the LST with a coefficient of
determination of $R^2 = 0.65$ and regression Equation 4. Thus, with an increase in LST, NDVI of the delta decreased as also reported by Yue et al. [11]; Huang and Ye [22]; and Dong et al. [6].

$$LST = -19.034 \times NDVI + 34.14$$

(4)

Yue et al. [11] also reported an opposite relation between the LST and NDVI in the Shanghai city of China. Choudhury et al. [2] have also reported the decreasing trend of vegetation with respect to increasing LST in Asansol-Durgapur area of the West Bengal. Dong et al. [6] explored the relationship of LST with NDVI in the Karst area and identified the opposite relation between them. Sun et al. [23] have reported a significant reduction in LST in the areas nearby water bodies, lakes, etc. in Beijing, China. Choudhury et al. [2] examined the association between LST and deriving factor such as NDVI of Asansol-Durgapur Development Region and found an inverse trend between these parameters.

Rasul et al. [24] reported that climate change is an established fact affecting food, fresh water, agriculture, natural ecosystems, health, biodiversity and socioeconomic sectors around the globe. The increase in the global temperature was recorded as 0.76 °C during the last century but in the first decade of the 21st century 0.6 °C rise has been noticed [24-25]. Pakistan is vulnerable to climate change because of its location in a geographical region where the temperature increases are expected to be higher than the global average [24-26]. Being in the arid and semi-arid region, it largely depends on the river irrigation system which is mainly fed by the Hindu Kush- Karakoram-Himalayan (HKH) glaciers which are reported melting rapidly due to global warming. Thus, the country will face risks of variability in monsoon rains, floods, and extended droughts.

4.4. Relationship between NDVI and Wheat Crop Yield

Based on the data of wheat crop yield of the Delta of different years, a statistical relationship between the wheat crop yield and NDVI of the delta was developed as shown in Figure 7. It shows a linear and positive relationship between the two variables, and it is described by the regression Equation 5 with a coefficient of determination of $R^2 = 0.89$.

$$Yield \text{ of wheat crop} = 3.241 \times NDVI - 0.3123$$

(5)
5. Conclusion

Variations in the LST of the Indus delta for the last 27 years (1990-2017) and its impact on the vegetative cover was analyzed. Altogether, a relationship between NDVI and the yield of the wheat crop was observed. The analysis revealed that on average, there was an increase of 1.74 °C in LST of the Indus delta during the last 27 years. The temporal variation in the NDVI of the Indus delta showed the highest NDVI of 0.725 in the year 2005 followed by the year 2010 with NDVI of 0.712. While the lowest NDVI of 0.545 was observed in the year 2017. There was a fair but negative statistical correlation between NDVI and the LST with a coefficient of determination of $R^2 = 0.65$. From NDVI maps, the decreasing trend in vegetative cover in the Indus delta is clear. A positive linear relationship between the NDVI and the yield of the wheat crop with a coefficient of determination of $R^2 = 0.89$ was observed. Such studies are essential for the guidance of policymakers for taking measures for mitigating adverse impacts of climate change on the environment.

6. Acknowledgements

We are highly grateful to the U.S.-Pakistan Centre for Advanced Studies in Water (USPCAS-W), Mehran UET, Jamshoro, Pakistan for funding the project “Assessing Impact of Seawater intrusion on soil, water and environment in the Indus delta using GIS and RS tools”. We are also thankful to the USGS (United States Geological Survey), Crop Reporting Services (CRS) Sindh, Google Earth for providing the necessary data.

7. Conflict of Interest

The authors declare no conflict of interest.

8. References


Readiness for E-Tendering in the Construction Sector—Designing a Computer Programme

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Abstract

Development of a country is measured by the number and quality of modern and contemporary projects that have been and are being established. As the construction industry is the nucleus for the development of any country, the stages of each project are tracked and monitored. It was found that the procurement stage has the biggest and most important influence in the successful completion of the project with the desired results. This research aims to eliminate corruption in the procurement process, identify the additional factors relating to a contractor’s qualification that contribute towards an increase in the quality of the project; designing a computer programme that conducts the tender process electronically to avoid any human contact. The researcher designed a questionnaire which contains a number of factors that would increase the efficiency and quality of the project. The researcher distributed 50 questionnaire forms and received back 46 completed forms. The questionnaire outputs were analyzed by using the SPSS software which can be defined as a software package used in statistical analysis for data. After analyzing the results a nominal group session was held. This consisted of eight employees with technical, financial, legal, and supervisory and IT expertise. The work was collective and many questions were asked. All relevant factors were discussed. It was agreed to cancel three factors only as being irrelevant to the contractor’s qualification process. The most significant findings were that if the organizations adopt the E-T system in the tendering process, corruption cases will disappear, the tendering process will be achieved with high level of integrity and transparency, and in order to implement the E-T system, the organization must be ready to change, the employees should have enough courage to adopt the system, and there would be a need for at least one person to play the role of champion/leader.

Keywords: Computer Programme; Corruption; Electronic-Tendering; Electronic-Procurement; Joint Venture; Standard Bidding Documents (SBDs).

1. Introduction

The construction industry is classified as an information-intensive industry and is described as one of the most important industries in any developed/developing country that is undergoing a period of rapid change. Now a days, the construction industry is also acclaimed as having a great lack of knowledge and awareness about information and communication technology (ICT) and innovative information and web-based communication processes, systems and solutions which may be useful in the procurement, life cycle and delivery of projects [1]. From 2005 to 2018, through the monitoring of the Government Contracts Department for projects and supervising of the contracting process, the defects in the contractual processes were noticed at several points. Consequently, many methods have been restored in solving these problems: the 1st solution was issuing instructions to execute government contracts, followed by issuing

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Standard Bidding Documents (SBDs), and finally, the establishment of a specialized website for government contracts whose main task is to deliver information in a transparent and equitable manner to all contractual parties.

Publishing on a site in Iraq is similar to the global site DG Market and all government parties are obliged to publish their tenders on this site, and therefore, all interested (registered parties) whether an individual, entity, or a coalition, can view the tender and submit a bid for it. Thus this will have a positive impact in investment because the existence of a healthy contractual environment will generate an open investment area that will be easier to manage, and the projects will yield the best results.

The electronic portal project was not suddenly conceived, but was planned taking into consideration the experiences of other countries, thus benefitting from this. Some of them, such as Korea and Malaysia, were successful and had a leading role in the field of electronic procurement, while some, like Saudi Arabia, faced, like Saudi Arabia, faced failure or had to stop at a certain point and could not complete the project. All government parties will be obliged to implement the contract on the portal while also observing the traditional method meaning that the publication in the newspapers will not be canceled, and purchase of the tender documents would be by the bill only. The existence of corruption cases, absence of transparency and accountability, poor quality of tender evaluation, the delay due to complex processes of the traditional methods, and the need to utilize innovative technologies to gain greater efficiency and cost savings throughout the institutions are the main reasons and justifications for undertaking this study.

The value of this study lies in many practices on e-tendering and their impacts will be discussed in this research. The ministry of planning will benefit from the results of the study, and obtain an idea of how its institutions can effectively manage the portal to improve their performance. This study will provide a competitive advantage for the companies. The research presents additional criteria to choose the most appropriate contractor, and these criteria are flexible and can change or be less or more according to the project. The results of the research were applied in a software programme; this software will be adopted in the ministry of planning to be as a practical application of the portal project.

2. Literature Review

This world is continuously changing and uncertain. The construction industry and its participants have to be creative, needing to have enough understanding of the changeable environments, noticing the opportunities, and increasing the existing confidence level for its capability to acclimate. Sharing the information of the project electronically - from inception, design, through construction, and into operation of the project - can be the guide to great efficiency being gained for all concerned parties [2].

Several studies have been conducted on the process of transition from paper to the electronic system. Mahidin et al. (2005) conducted a study on the impact of adopting e-tendering in their contracts. They concluded that the document flow process was reduced approximately from 73 to 30 days. Other benefits involved the reduction in printing and copying costs, halved the total costs on telephone and faxing volumes between the owner and contractors [3], while Kassim and Hussin (2013) concluded that use of electronic system explains nearly 69% of the variance in transparency, 87% in service performance, 79% in efficiency and 67% in information quality. Thus, it can be concluded that the highest success in achievements is on the improvement of service performance, followed by efficiency, transparency and information quality. The results also show that user attitude has a direct influence on the system usage [4].

Oyediran and Akintola (2011) conducted a survey about e-tendering in Nigeria, and they state that the few professionals who have participated in E-T have experienced a low level of savings in costs, and a moderate level of savings in time, and there is need to develop a capacity building knowledge backbone to drive the adoption of E-T [5]. In Australia, Neupane and Yong (2012) conducted a study which found that transparency and accountability are the most important benefits from public e-procurement. Other benefits include increasing competition among bidders, best quality of work and services, and increasing more consistency in government procurement, which helps governments to reduce corruption in public procurement. Most importantly, the study finds that most of the developing countries’ government missions and objectives of adopting e-procurement technology are to increase transparency, accountability, real-time access information, and increase competition among bidders, which ultimately reduces corruption in public procurement [6].

Bulut and Yen (2013) studied e-procurement in China and found that the perception and understanding of e-procurement among decision makers and other key stakeholders, in addition to political and institutional support, are prerequisites for E-P implementations and yet are not the only factors that lead to success. And the level of development of E-P within the public sector does not necessarily correlate with the level of development of these countries [7]. Dominic (2014) aimed to establish the e-procurement practices employed by Kenya Revenue Authority (KRA), and to establish the factors influencing application of e-procurement at KRA. He arrived at the result that understanding the E-P concept was a little difficult for organizational stakeholders such as senior management and end-users, a fear of making errors, a lack of confidence, lack of technology and innovation champions within the organizations, In terms of factors,
he found that the factors considered to affect E-P implementation at KRA include organization readiness, the size of the firm, trust, and risk [8]. KRA. He arrived at the

In Bangladesh, Rahim (2014) intended to evaluate the progress of implementation of the e-tendering system. He found that the implementation of the E-GP system has been done very successfully withstanding all the possible challenges. They were very sincere and motivated in doing that. They are giving training to the staff and simultaneously implementing the system, at the same time 99% respondents desired the E-GP system for the free, fair and transparent administration and participation in tenders. But 87% of the respondents needed E-GP training either in refreshing or fresh training courses. Among the respondents, 63% were not satisfied with internet browsing speed, and 28% were not satisfied with bankers support services [9]. In the USA a study had been conducted by Altayyar and Beaumont-Kerridge (2016). The study aimed at investigating and examining the barriers that affect the adoption of e-procurement in four Saudi Arabian SMEs. Weak infrastructure and lack of government support were found to be the most challenging barriers, and quantitative data further established that there was a lack of procurement, specific laws, and mistrust in electronic fund transferring mechanisms by the available options [10].

Pavithra et al. (2018), analyzed the effectiveness and challenges of the E-T marketing system. The significant findings were that the E-T system has considerable potential to increase competition and transparency in agricultural markets, and to reduce costs of trade for both buyers and sellers without negatively affecting their trade relations and revenues. Farmers benefit from E-T, and there is a need to create awareness of the benefits of e-trading among farmers, and build their capacity in online banking and grading of produce at farm level. E-T is successful in larger markets, but not in smaller ones. This is because of several issues related to capacity of the market committee, and fear among traders that with automation they would lose their business to large traders [11].

In Kenya, Nurwin (2018) has determine the effect of regulated electronic tendering on the implementation of preference regulations. The researcher found that tendering processes are accomplished electronically, employees prepare tender specifications electronically, and suppliers are also able to bid electronically. As a result, paper-based transactions are reduced and the marginalized groups are empowered. There is thus need to do all the tendering processes online. This will also bring about a speedy exchange and accessibility of information, ensure efficiency in service delivery and, overall, improve firm performance [12]. In Australia, Al-Yahya et al. (2018) developed a conceptual model to assess the E-T readiness in a construction organization prior to implementation, explore the current level of E-T readiness in construction organizations, and build a theoretical model to assist in the E-T readiness in any construction organization's E-T implementation. The data is collected and refined, validity and reliability of the model assessed by the Delphi method. The search result was that there is a lack in empirical support of the conceptual model for validation, the model contains five themes: People, Process, Work Environment, Technology and Service Providers, and the Service Providers theme with its structures (communication, market and technical) is proposed as a necessary support for successful E-T implementation [13].

3. Tendering

Tendering has been considered one of the methods for awarding government contracts (the fairest way) and the most likely way that ensures a favorable outcome of the Government’s spending on the general budget [14]. Many types of tenders have been listed in Table 1.

Table 1. Comparison between types of tendering in international countries and Iraq

<table>
<thead>
<tr>
<th>Country</th>
<th>Major types of tenders to be used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nigeria</td>
<td>Open tender</td>
</tr>
<tr>
<td>Malaysia</td>
<td>Restricted tender (selective tender)(limited tender)</td>
</tr>
<tr>
<td>Turkey</td>
<td>Negotiated tender.</td>
</tr>
<tr>
<td>Kenya</td>
<td></td>
</tr>
<tr>
<td>India</td>
<td></td>
</tr>
<tr>
<td>Iraq</td>
<td>1- One Stage Bidding (nine methods)</td>
</tr>
<tr>
<td></td>
<td>• General Tender</td>
</tr>
<tr>
<td></td>
<td>• Limited Bidding</td>
</tr>
<tr>
<td></td>
<td>• The general tender in the form of technical qualification</td>
</tr>
<tr>
<td></td>
<td>• General tender in two stages</td>
</tr>
<tr>
<td></td>
<td>• Direct invitation</td>
</tr>
<tr>
<td></td>
<td>• One bid (the only offer)</td>
</tr>
<tr>
<td></td>
<td>• Direct contracting</td>
</tr>
<tr>
<td></td>
<td>• Direct purchase from the manufacturers</td>
</tr>
<tr>
<td></td>
<td>• Procurement Committees</td>
</tr>
<tr>
<td></td>
<td>2- Two Stage Bidding</td>
</tr>
</tbody>
</table>

1766
3.1. Traditional tendering processes

1. Internationally: Usually the government is considered as the major client in construction sector contract. A client has been defined as the owner of the project. A contractor has been defined as the provider or tenderer. A consultant has been defined as information broker.

The process starts when the client has an intention to undertake a construction project. The client hires a consultant to prepare the tender specification by making a feasibility study. Traditionally, the first step is the declaration that is made by the client over a specified period through printed means like newspapers, the website, or public media. After that, the contractor who wants to submit the bid will buy the documents of the tender, insert the information required, and presents the finished tender prior to the final date of submission, as is shown in Figure 1. The grey boxes represent the main phases in tendering:

![Figure 1. General tendering processes in the construction industry according to client, consultant, and contractor perspectives [14]](image-url)
2- **Locally:** Figure 2 is a flowchart that illustrates the traditional tendering process locally (in Republic Of Iraq). The researcher obtained it from the instructions of implementation in government contracts in Iraq, and developed it as shown in the flowchart:

![Flowchart of traditional tendering process locally (in Republic of Iraq)](image)

3.2. **Electronic Tendering**

Our world today presents no technological barrier – the only likely barriers being the people and the processes themselves, – using ICT in practice and daily work. The National E-T Imperative (NETi) is a national initiative that has merged every action or component of the whole construction tendering into an electronic or digital medium hoping that it can overrun time, economical, geographical, and people-based errors and inefficiency barriers. Thus the process will be faster, more profitable and more efficient for all the participants in this industry [15].

E-T is the process of sending a demand for information and costs to vendors and receiving a reply using the technology of Internet [6, 16].

E-T is basically a term used to describe the publishing and receipt of tender information, receiving of tender documents, a signal of interest in tendering, acceptance of the tender amount and finally choosing of a successful tender through the Internet [5].

E-T is an IT tool that has been highlighted by the construction industry experts to support changing the culture of this industry and improving its operation [17].

Some benefits derived from the application or adoption of electronic tendering in the procurement process are:

- Provide greater opportunities for small and regional firms, and provide improved and safe access to tender information [1, 18].
- Provide remote accessibility to the system [5].
- Better status for monitoring and tracking of applications can be provided by E-T, and direct human contact will be eliminated in tendering and other works and services; internal efficiency to increase in government departments, and corruption will decrease significantly [6].
- Reduces the cost of printing and copying which saves time and resources [17]; up to 90% in preparing, copying and distributing tender documents [19].
- Centralize the tendering process and documents, which allows for easier access to tender documentation by all parties [19].
- E-T system has minimized the tender life cycle by about 43 days, and this results in increasing the document flow speed by about 58.5% [3].

While the risks that can result in the application of E-T is uncertainty, reducing staff, changes in relationships and daily work, create resistance against that change, so it is necessary to recognize champions that act as individual change agents in the team and in the organization [20]. The other risk is that the essential changes in the process may break
existing organizational processes or stopping the organization. Insertion of modern technologies will result in changes. Employees might have to change the way they work to follow-up technology which might led to inefficiency in work, no motivation, disorganization, and some may fear for their job safety [2].

Using new IT solutions, such as e-procurement and e-tendering, represents a strong stimulation to move from a bureaucratic model of administration (based on standard procedures, only committed to rules), to the virtual bureaucracy in which communication is informal and electronic; employees are multi-functional; jobs are enriched in content and “limited” not only by the expertise of the employees, but also by the extension and sophistication of the mediation offered by technologies [21].

4. Statistical Analysis of The Questionnaire Data

The questionnaire outputs were analyzed by using the SPSS which has been defined as a software package used in the statistical analyzes of data. The results of the analysis are shown in Table 2.

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Factor</th>
<th>N</th>
<th>Mean</th>
<th>Std. deviation</th>
<th>Cronbach's Alpha</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Contractor's compliance with technical conditions and specifications</td>
<td>46</td>
<td>4.78</td>
<td>0.467</td>
<td>0.984</td>
</tr>
<tr>
<td>2</td>
<td>Obligation of the existence of the project implementation manager</td>
<td>46</td>
<td>4.63</td>
<td>0.572</td>
<td>0.983</td>
</tr>
<tr>
<td>3</td>
<td>Awareness about the existence of the specialist engineer with the team</td>
<td>46</td>
<td>4.59</td>
<td>0.652</td>
<td>0.983</td>
</tr>
<tr>
<td>4</td>
<td>Finance situation</td>
<td>46</td>
<td>4.59</td>
<td>0.652</td>
<td>0.983</td>
</tr>
<tr>
<td>5</td>
<td>Final calculation for the last three years</td>
<td>46</td>
<td>4.57</td>
<td>0.655</td>
<td>0.983</td>
</tr>
<tr>
<td>6</td>
<td>The existence of a system for the management of the project</td>
<td>46</td>
<td>4.54</td>
<td>0.585</td>
<td>0.983</td>
</tr>
<tr>
<td>7</td>
<td>Achievement of previous projects on time</td>
<td>46</td>
<td>4.50</td>
<td>0.548</td>
<td>0.984</td>
</tr>
<tr>
<td>8</td>
<td>Complete the execution of contracts similar to the work successfully</td>
<td>46</td>
<td>4.48</td>
<td>0.722</td>
<td>0.983</td>
</tr>
<tr>
<td>9</td>
<td>Adoption of the work progress program</td>
<td>46</td>
<td>4.43</td>
<td>0.655</td>
<td>0.983</td>
</tr>
<tr>
<td>10</td>
<td>Adoption system of risk forecasting and risk management</td>
<td>46</td>
<td>4.26</td>
<td>0.743</td>
<td>0.983</td>
</tr>
<tr>
<td>11</td>
<td>Achievement of previous projects within the budget assigned to them</td>
<td>46</td>
<td>4.24</td>
<td>0.639</td>
<td>0.983</td>
</tr>
<tr>
<td>12</td>
<td>Specialized experience (particular period of years of work in the required specialization)</td>
<td>46</td>
<td>4.24</td>
<td>0.603</td>
<td>0.984</td>
</tr>
<tr>
<td>13</td>
<td>Existence a program for a quality policy in previous projects</td>
<td>46</td>
<td>4.24</td>
<td>0.705</td>
<td>0.983</td>
</tr>
<tr>
<td>14</td>
<td>Adoption of health and safety policy</td>
<td>46</td>
<td>4.22</td>
<td>0.696</td>
<td>0.983</td>
</tr>
<tr>
<td>15</td>
<td>Contractor's current status (workload and ability to take over a new project)</td>
<td>46</td>
<td>3.98</td>
<td>0.649</td>
<td>0.984</td>
</tr>
<tr>
<td>16</td>
<td>Contractor has have his own mechanical equipment</td>
<td>46</td>
<td>3.98</td>
<td>0.745</td>
<td>0.983</td>
</tr>
<tr>
<td>17</td>
<td>Health and safety indicators appear in previous project registration.</td>
<td>46</td>
<td>3.89</td>
<td>0.849</td>
<td>0.983</td>
</tr>
<tr>
<td>18</td>
<td>The importance of having a field laboratory</td>
<td>46</td>
<td>3.74</td>
<td>0.773</td>
<td>0.984</td>
</tr>
</tbody>
</table>

Valid N (listwise)                                                                 46

Four questionnaire forms were filled with feedback. The 1st one commented on Factor 5 (making sure that the company is sober and not losing); Factor 13 (preference for bidding); and Factor 14 (contractual condition and within the tender). The 2nd one commented on Factor 11 (as requested by the employer or beneficiary party). The 3rd one commented on Factor 16 also (no stipulation that the equipment should be owned by the contractor, but requiring a rent contract for needed equipment to cover the contractual period for the implementation of the project). The last one commented on Factors 6 and 10 (it is preferable to be a governing criteria at present and in the future); Factor 16 (equipment is important, not necessarily owned but can be rented). And Factor 13 (very important for the present and in the future).

The authors were of the opinion that the factor which obtained the mean less than 4 be cancelled. Firstly, the Factors 1, 2, 3, 4, 5, 7, 8, and 12 are constant (governing) criteria required in the small SBDs. Secondly, the Factors 6, 9, 10, 13, and 14 have values with means more than 4 and are not required in the constant criteria, so the authors adopt them as a criteria required in the computer programme in addition to the constant criteria, they have been added as flexible criteria in which the employer can change according to the nature of the project. Finally, the factors 15, 16, 17, and 18
have values with means less than 4, so the authors decided to cancel them, but before cancelling them the researcher preferred to conduct more than one method to filter the factors and conducted a session of the nominal group technique to apply rankings to the factors.

The authors showed the factors to the sample for the purpose of ranking the factors that were put forward in the questionnaire. The sample agreed that the second group with the mean above 4 must be adopted in the computer programme, and agreed to cancel the factors that obtained a mean under 4. However, the sample were not in agreement and rejected the cancellation of Factor 16 because it is required in the small SBDs as a constant (governing)criteria and requested to adopt it despite obtaining mean of 3.98 < 4.00.

5. Computer Programme

The programme consists of four stages which are divided into constant and variable parts. The constant part which is constant for all projects and tenders, involves the second stage; the first, third, and fourth stages are the formation of the variable part which varies from one project to another.

![Figure 3. Components of the Programme](image)

5.1. Programme Methodology

The programme consists of four general steps which are:

- Information about the tender such as the project name, tender number, type of budget, the name of the contracting party and the e-mail address of the responsible official, etc.
- The constant criteria mentioned in the small SBDs, when the user passes it, he will be moved to the next step.
- Third step is the flexible criteria that varies from one project to another and according to the request of the employer.
- The report of awarding to the most appropriate tender.

Before explaining about the program, let us have a simple clarification about SBDs:

**Standard Bidding Documents**

The SBDs are regulatory procedures for organizing the procurement process, which are adopted for obtaining unified and easy-to-filter tenders, and obtain the best in terms of technical offer and the lowest in terms of financial offer. The standard documents are 18 documents, each document used for a specific type of project.

In the case of projects for the construction sector, the public works document is used a lot, which is divided into three types of sub-documents:

- Works document for large projects: for projects with a budget of 10 billion Iraqi dinars (ID) and above.
- Works document for medium projects: for projects with budget of 5-10 billion ID.
- Works document for small projects: for projects with a budget under 5 billion ID.
In this research, the researcher adopted the third type of documents, the small document consists of 8 parts, and Table 3 presents a brief definition of each part:

<table>
<thead>
<tr>
<th>No.</th>
<th>Part Name</th>
<th>Part Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Instructions for bidders</td>
<td>This part is constant, unchangeable or modified.</td>
</tr>
<tr>
<td>2</td>
<td>Tender data sheet</td>
<td>This part contains texts that are filled by the employer (the contracting party).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Employer's requirements clarified in this part, the texts that confuse the employer (in the 1st part) can be treated in this part.</td>
</tr>
<tr>
<td>3</td>
<td>Evaluation and qualification criteria</td>
<td>Fixed criteria to be filled by the employer for the purpose of evaluating the bidders and selecting the most efficient among them, the filling of these criteria varies according to each project and its requirements.</td>
</tr>
<tr>
<td>4</td>
<td>Tender forms</td>
<td>The only part that is filled by the bidder. The bidder takes the forms from the 5th part, fills them and places them in this part, puts all the information such as bill of quantities, staff, equipment etc.</td>
</tr>
<tr>
<td>5</td>
<td>Requirements for works</td>
<td>This part is filled by the employer, it is the requirements of employer from the bidder, such as drawings, bill of quantities, schedule of work for auxiliary items, reserve amount. The bidder fills the fourth part based on this part.</td>
</tr>
<tr>
<td>6</td>
<td>General Conditions of Contract</td>
<td>This part is constant, unchangeable or modified.</td>
</tr>
<tr>
<td>7</td>
<td>Private Conditions of Contract</td>
<td>This part is filled by the employer, The general conditions are dealt with while the employer can establish his own conditions for the project in this part.</td>
</tr>
<tr>
<td>8</td>
<td>Standard Models</td>
<td>This part is non-obligatory, where the employer can be guided by it or left it according to the need.</td>
</tr>
</tbody>
</table>

5.2. The Fundamental Components for the Stages

1st stage components are:

1- Information about the project and the owner (contracting party).
2- Information about the tender.

2nd stage components are the criteria that are requested in the SBDs for small works which are to be filled by the owner according to his requirements under the limitation of the tender, and these criteria are:

- Financial situation

The financial situation of the bidder will be clarified by Table 4 which shows the amount of money required to be held by the bidder to be able to undertake the proposed project.

<table>
<thead>
<tr>
<th>Subject</th>
<th>Request</th>
<th>Single company</th>
<th>Joint venture all partners</th>
<th>Joint venture every partner</th>
<th>Joint venture one partner at minimum</th>
<th>Submission requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>The cash</td>
<td>The bidder have to provide cash of (<em><strong>) ID within a period of (</strong></em>)</td>
<td>Requirements must be met</td>
<td>Requirements must be met</td>
<td>Requirements must be met by (___)</td>
<td>Requirements must be met by (___)</td>
<td>Under the cash flow statement*</td>
</tr>
</tbody>
</table>

- The experience

The experience of the bidder must be as outlined in Table 5.

<table>
<thead>
<tr>
<th>Subject</th>
<th>Request</th>
<th>Single company</th>
<th>Joint venture all partners</th>
<th>Joint venture every partner</th>
<th>Joint venture one partner at minimum</th>
<th>Submission requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specialized expertise</td>
<td>Participate as a contractor, contract manager, or sub-contractor in (<em><strong>) contracts for the previous (</strong></em>) years with a minimum amount of money (___) ID for contracts executed for similar works and have been completed in full quality successfully.</td>
<td>Requirements must be met</td>
<td>Requirements must be met</td>
<td>Not required</td>
<td>Meet</td>
<td>Under the specialized experience form</td>
</tr>
</tbody>
</table>
The bidder has to provide the details of the employees proposed for work in the execution of the contract, specifying their previous experiences, according to the forms.

**Equipment**

The bidder must prove ownership or the acquiring of major equipment; this equipment is presented as a table containing (for each equipment) equipment type, specifications, and required number.

The bidder has to provide the additional details of the proposed equipment to be used in the execution of the contract under the tender form.

3rd stage components are the factors that have been derived from the questionnaire which are:

- Adoption of the work progress programme.
- Adoption system of risk forecasting and risk management.
- Existence a programme for a quality policy in previous projects.
- The existence of a system for the management of the project.
- Adoption of health and safety policy.

4th stage component is the report of awarding which will be sent to all submitters.

It is important to mention here that the criteria in the second stage are the standards adopted by the Ministry of Planning and other Iraqi ministries, while the factors in the third stage are research material proposed by the researcher and have been added to obtain the highest quality of the project and to choose the best offer and be flexible according to the nature of the tender and according to owner opinion and his request. After the evaluation of the programme, it will be adopted in the Ministry of Planning.

5.3. Basic Principles of the Programme

- This programme has been designed especially for projects with a budget less than five billion (5000000000) ID, for projects in the construction sector.
- The fake or incomplete tenders will be automatically rejected and excluded in the 1st stage.
- The success or pass of any tender depended on the fulfilment of the criteria and factors which have been derived from the technical offer.
- After the passing, the programme has to choose one offer from many successful tenders, and the tender that is chosen will be the tender with the least financial offer.

6. Conclusion

The most important conclusions are:

- To implement the E-T system, the organization must be ready to change, the employees should have enough courage to adopt the system, and there is a need to have at least one person to play the role of (champion).
- In order to obtain projects completed with highest efficiency, a number of factors were added to be met by the contractor. These factors are placed in the SBDs (exactly in Part II: Tender Datasheet, Additional data to be submitted by the bidder) as part of the owner's requirements which must be provided, and after crossing the ruling criteria and achieving them.
- The characteristics of the computer programme and its terms of reference have been developed and written in cooperation with the Ministry of Planning, which will implement the project soon, and this confirms that the data that were written are real, accurate and realistic with the reality of the current situation.
- By taking the opinions of experts during the nominal group session, many facts have been discovered, the most important is that when the organization adopts the E-T system in the tendering process it will reduce the average price of the tender.
- It has been proved that the transition to the E-T system reduces the cost by converting to an electronic system instead of the paper system, and reduces the time consumed because the system works 24 hours a day and has a remote access feature.
7. Funding and Acknowledgement

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8. Conflicts of Interest

The authors declare no conflicts of interest.

9. References

Nonlinear Deterministic Study of Seismic Microzoning of a City in North of Algeria

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Abstract

This paper presents also an overview of seismic microzonation studies of the city of Mohammadia-Algiers, which are important for a detailed ground movement modeling of urban cities. According to the seismic history of the city, one extraordinary earthquake event has been taken into consideration is Boumerdes earthquake (Algeria, May 21, 2003, magnitude Mw=6.5), that caused a huge damage. Thereby, the variability prediction of the seismic ground movement in a given built-up area, it is considered as an effective tool for planning appropriate urban development and understanding both seismic risk and damage pattern, caused by a strong movement event. We note that the shaking level is mainly described in terms of both maximum ground acceleration and visualized amplification by using response spectra. The study is carried out in two steps: - a detailed mapping of the geology and geotechnical properties of the area - numerical modeling of expected ground motions during earthquakes. A qualitative microzonation of the Mohammadia-Algiers city is presented, and it is discussed by comparing it to the historically reported damage of the 2003 Boumerdes earthquake. Finally, this study deals with the seismic microzonation map development, based on a SIG geological model.

Keywords: Seismic Microzonation; Site Effects; Shear Wave Velocity; Ground Response Analysis.

1. Introduction

The town of Mohammadia city is located in the heart of Algiers, about 10 km to the east. It is bounded, on the west, by Oued El Harrach, on the north, by the sea, on the south, by the national road (NR.5) and, on the east, by the municipalities of Bordj El Kiffan and Bab Ezzouar. It is located according to the following geodetic coordinates 36° 44' 00" North 3° 08' 00" East (Figure 1). This zone is known by its great seismic activity due to the approximation of the Eurasia and Africa tectonic plates. Algeria's north has witnessed several destructive earthquakes whose the majority has been registered. To illustrate, the Setif earthquake (419) which is the first historically known list, postponed by Miniati (1995) [1], then Algiers in 1365 and 1716, Oran in 1790, and Gouraya in 1891. In the recent period, we can list the cheleff / (The city of Orleans) earthquakes on September 09th, 1954, El Asnam on October 10th, 1980, M = 7.3 [2 -4], Constantine on October 27th,1985, M = 6 [5], Tipaza on October 10th, 1986, M = 6.1 [6] Mascara on August 17th, 1994 [7], Ain Temouchent on December 22nd, 1999 [8], Beni-Ouargilane on November 10th, 2000, M = 5.7 [9] and Boumerdes on May 21st, 2003, M = 6.8 [10-12].

The crust quake is the reason for the majority of damages, which are generated by an earthquake. Therefore, they can represent the direct repercussions of soil vibrations coming directly from the focus. However, lands movements can be

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created at the same time and cause other types of material damage. One of the most important destructive factors is related to the tremor duration, which depends on the soil constitution. According to the underground topography of the site and the soils nature, seismic tremors, as well as the caused damages, can be either diminished or amplified.

For identifying the areas with a high seismic risk and guarding against the damage that may occur, we try through this method to estimate the lands movement’s parameters or the macroseismic intensities during an earthquake triggering. The seismic movement arrival, in a fixed site, is estimated through the seismic hazard; it may suffer from variations in terms of value [13]. That is to say, even if the movement from the seismic source is attenuated in some places during the perturbation spreading, the morphological and topographical characteristics induce amplification [14-18]. Moreover, the movement interaction with the geotechnical and hydrogeological conditions which are really related to a fixed site can create some induced phenomena such as the liquefaction phenomenon, landslide, etc.

Studying seismic hazard for a good design and efficient structures implantation, an accurate assessment of earthquake-resistant amplification is required. The tremor level is mainly described in terms of maximum ground acceleration and amplification [19] and preparing microzonation maps will assure, in a way, an effective solution for urban planning in terms of seismic risks [20, 21]. On the other hand, the cities microzonation will allow the characterization of a potential seismic vulnerability as well as the risk, which must be taken into account, when to design new structures or modernize those which have already existed. Rational approaches for seismic microzonation have been proposed in a very large number of journal and conference papers [22-28].

Seismic microzonation is required in urban areas or the ones which are close to them, which belong to the high seismic risk area; it is mainly based on identifying the seismic activity sources [29]. In the region when to study microzonation of Mohammadia city, 1D dynamic analyzes were carried out through the field data which contain geological, geophysical and geotechnical surveys for predicting. The analysis of the movement response is applied on sub-basically movements, in order to prepare the design spectra response and, finally, determine the ground responses that include the dynamic insistence and the seismic zoning map [30, 31]. Local site effects play, as well, an important role in the resistant earthquake design and should be separately treated as concerns each case [32]. The site factors can affect the earthquake waves in two ways: Topographic effects that are resulted by changing the nature waves (such as the amplitude and the satisfied frequency of ground movement) as well as the effect of the soil layers number and type on the bedrock [33]. Moreover, urbanization is a factor that foresees the construction of the megastructure. The main reason, for human and material damages, is when the desired importance is not taken into account by preparing adequately for a possible consideration of the seismic hazard.

The seismic zoning map presents a large scale view that allows an interpretation of the site behavior during an earthquake because of the local variations in the soil type and geology. Earthquakes are unavoidable but it is possible to minimize their consequences if the areas, which are the most sensitive to soil movements and suffer from a maximum action, are identified. Seismic Microzonation will be a response to the need for cushioning measures against seismic risks, as it gives a real response in terms of soil movement with a higher resolution [34-35].

In this study, the microzonation analysis of the seismic risk, according to the effect of Mohammadia city, in the province of Algiers, is explained.

Figure 1. Satellite image (Google Earth) of the locations of Mohammadia city
2. Microzonation Methodology

The site effects are considered as an important challenge in the seismic prevention. In this context, it is obviously not possible to anticipate an earthquake, but it seems admissible to predict where and how the seismic signal should be amplified [36]. Therefore, the microzonation goal is to delimit, on maps, the zones with a homogeneous hazard. For each zone, we must identify the fundamental sites frequency and recognize the spectral site’s response (their transfer function). Therefore, the soils recognition and their response allow the identification of typical behaviors and the adaptation of the structures design spectra, which are imposed by the seismic regulations, in order to take into account the site effects [37-38]. In fact, regulations can only give a lump and simplified values that rarely reflect the soils physical reality. The challenge is to understand the phenomena as well as to develop reliable methods for refining the regulation, either in a national context or in the local one (Within the framework of the risk prevention plan).

On the whole, it is important to estimate the response of soil layers, under the seismic excitations, as well the variation of soil earthquake movement characteristics in the soil surface. The general methodology, to proceed to the seismic microzonation of Mohammadia city, has been subdivided into four major elements:

1. The assessment of soil movements’ parameters through historical seismicity and earthquake movements’ data that include the position of potential sources, magnitude, mechanism, distances to the epicenter.
2. The site characterization by using geological, geomorphologic, geophysical and geotechnical data.
3. The assessment of local site effects, including amplification, predominant frequency, the risk of liquefaction, landslides, tsunami, etc.
4. The preparation of seismic microzonation maps.

The method, which is used to assess the 1-D seismic response of the site, is a simple computer-based analysis according to the seismic response of soil profiles. The seismic response determination of soil profiles, with a seismic stress, is related to the assessment of the seismic movement characteristics (displacement, speed, acceleration, stress) in the vicinity of the free surface, see (Figure 2).

3. Geology and Geomorphology of the Study Area

The geological context of Algiers is very complex, as there is a sudden passage from the ancient metamorphic soils of the primary age to the tertiary-age of sedimentary soils. They have included three major elements (The crystalliferous shelf, the tertiary and the quaternary), (See Figure 3).

Ancient soils are highly tectonized; they consist of crystalliferous rocks which are mainly composed of gneiss, schist, mica schist and marbles. The crystalliferous shelf is composed of the metamorphic rocks of the Algiers massif which flocks from Baïnem to the Agha, as well as three points, which are isolated by the Neogene from the west in Ain-Benian, Sidi Fredj and from the East near Tamentfoust. As it is referred, according to certain authors, to the primary stage, it doesn’t present any element allowing the situation of these facies sedimentation in this era. However, the metamorphism age is very controversial. It is anticambrian for someones, Hercynian and even alpine for others.

According to several studies [39-40], it is observed, in the tertiary one, a stratigraphic gap coming from the Eocene and the Oligocene. The Tertiary is so represented by the Mio-Pliocene post-sedimentary forming, which cover the metamorphic facies and the metamorphic shelf; the latter is represented by coastal sandstones, on a conglomerate base.

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The Pliocene is composed of two layers which are: the Pliocene onset Pleistocene when the sea occupied the whole zone. This sedimentation through receives all the clay deposits, which would later compose the blue marls, with an average thickness of 200 m. This forming stands, in discordance, on the sandstones and Miocene marls.

In the plain of El Hamma, the Pleistocene is covered by a recent deposit. It forms the substratum of the Mitidja subsurface basin. On the surface, the marls show weathering and are observed on the outcrops of Algiers heights. In-depth, the marls are compact and compose sometimes conchoidal fractures. The passage from the Pleistocene to the Astienis is marked by the position of a glauconite green level which is clayey-sandy, with a thickness of 0.1m to 5 m, containing numerous grains of greenish glauconite and macro-fossils such as Terebratula, Ampula. The Astien, molassic facies with a thickness of 100 m to 150 m, is characterized by the following lithological:

- Marl-sandy facies;
- Calcareous-sandstone facies;
- Sandstone and sandy facies.
In Algiers, the Quaternary is composed of sandstone soils which are formed by sand, alluvium, scree, silt, and muddy clay. Its thickness can reach 30 m where we essentially distinguish the lower Pleistocene.

It consists of a detrital continental deposit, composed of red sands, forming the terraces that can reach up to 5 m thick. We also find marls and pebbles, in the Mitidja filling, which outcrop the southern Sahel piedmont. In Mitidja, it results from the erosion of the Tellian Atlas and the accumulation of products, which are generated during this erosion. The upper Pleistocene is spreading out the Algiers coast, in the Sahel in the form of terraces (See Figure 3).

The geological map 1/50000 of Algiers shows that the studied site of Mohammadia is located on an old alleviation flap, belonging to the stony clay series of the Mitidja. On the quaternary deposit, we find the recent series including sands which are, in a way, more or less clayish and, on the other hand, less rubified [40].

![Figure 3. A synthetic stratigraphic log showing more or less loose quaternary formations. These beds of variable thicknesses laying on solid formations with hard Pliocene, being able to represent the seismic bedrock](image)

4.1. Regional Hydrogeology

The Algiers region is characterized by several aquifers, which are the metamorphic complex aquifer, known by the presence of water in the metamorphic shelf, is located in the cracks, and fractures that are accumulated in the weathering zones. “ It appeared in the form of resurgences (sources) or aquifers “ The tertiary aquifer is the most important aquifer area in Algiers, whose the wall is composed of the Piacenzian blue marls, through an aquifer of Algiers’ drippy tableland, this aquifer is free, its water has been exploited for providing Algiers with drinking water. The Quaternary aquifer consists of Quaternary dunes sandstone. It forms aquifers in several towns of this zone:

- The littoral strip between Bordj El Kifane and the right bank of the Oued El Harrach;
- The eastern littoral plain, between the 1st of May city and the left bank of the Oued El Harrach;
- The littoral zone stretching between Ain Benian and Staouali heights;

In all these areas, the Pleistocene blue marls constitute the aquifer’s wall.

5. Characteristics of Algiers Seismicity

Algeria is situated on the Africa plate, which is in a perpetual collision with the Eurasia one. The collision of these two plates is proceeded with a speed of 5mm per year approximately; it gives to side of the border of the plate, a mountain
range, folds and faults, which are mostly orientated to NE-SW, NNW-SSE and shortening direction in the Tellian Atlas [41-45]. In our study zone, is that we mainly find out five (05) faults: Sahel fault, Chenoua fault, Blida fault, Zemmouri fault, and Thenia fault.

The Algiers seismicity is located on the northern fringe of the country; it is composed of morpho-structural fields (Tellian Atlas). The frequency and magnitude of the seismicity are important in the Tellian Atlas. However, it will diminish if we go to the south (Figure 4).

The seismicity of the northern of Algiers is characterized by superficial earthquakes; it is located on the first 20 kilometers. This seismicity is generally characterized by a weak to moderate seism. However, strong earthquakes occurred in the Tellian Atlas. We can list the major earthquake of El Asnam on October 10th, 1980 (Ms = 7.3) and the strong one of Boumerdes-Zemmouri on May 21st, 2003 (Mw = 6.8) whose the result of dead persons was estimated at 2286: 3323 wounded ones, 100 lost ones, 175,000 disaster victims and 18,000 destroyed homes. These results have shown to which extent the consequences of such events can be catastrophic on the socio-economic context. The active structures are generally composed of inverse and/or sliding faults (Figure 3). The faults, that generate these earthquakes, are mainly oriented to NE-SW [2, 4, 8, 10, 46].

6. Geotechnical Properties of Soils in the Region

The dynamic properties of the region soils were evaluated through the soil data that can be determined by dynamic or seismic tests, which are carried out either in the laboratory or in the soil [48]. Despite the fact that dynamic soil tests are extremely useful for the site characterization, these ones, whose the cost is not as high as one of the static penetration tests (SPT), are commonly used in the surveys field. Through the results of static penetration tests, the soils dynamic properties can comfortably be calculated according to empirical equations, which improve the calculation of dynamic resistance parameters of the ones of static resistance.

The standard penetration test (SPT) may be the most known and used in the world [49]. In addition to the large database of these tests and the financial obtained advantages; it is plausible to understand the soils dynamic properties through these tests. The parameters, that can be empirically deduced, are listed, such as the shear wave speed, shear modulus, and soil amplification. Several tests of the SPT type (52 tests of cored surveys), with a depth of 30m, were executed at the studied site of "Mohammadia" (Figure 5).
The SPT tests were systematically effectuated on the various facies, traversed by the cored surveys which are carried out in “Mohammadia” site. The results show the following lithological succession: A superficial covering that mainly consists of silts, sand, some consolidated passages and heterogeneous embankments per places, a layer of clay and / or marl which is localized in the surveys, a fine alluvium alternation which is essentially composed of Sand and silt with a few gravels, enrobed with a clay matrix, with roughly alluvium, in which we observe stones, pebbles, some of the blocks, as well as silt and gravels. These results reveal certain homogeneity of the site according to the compactness point of view.

In fact, the SPT tests values, registered over the first 30 meters, comply with the send level. The obtained raw values N1, N2 and N3 of the SPT test, of which only N2 and N3 will be taken into account, as the value N1 corresponds to the soil reworked part which is designed to elimination. We will, therefore, take into account the N2 and N3, as concerns the value of N. The main characteristics of the test are:

- The sheep weight must reach 63 Kg.
- The fall height is 76 cm.

6.1. Determination of the Shear Wave Velocity ($V_s$)

The $V_s$ measurements are possible in very stiff and gravelly soils. These parameters are obtained through the in situ direct measure, according to laboratory measures or empirical links with other usual geotechnical parameters, such as the SPT shocks number, a number of empirical links between $G_{\text{max}}$ and several parameters of the existing in-situ tests. However, they still need to be improved by collecting other data. The usefulness of such correlations is often limited by preliminary $G_{\text{max}}$ assessment. Therefore, the soil class assessment can be calculated through the average shear-wave velocity [50, 51].

Therefore, the soil class assessment can be acquired by using average shear wave velocities. The formula can be used to make a link between SPT counts and shear wave velocity, were calculated using (Equation 1) [52]:

$$V_s = 68.96 N^{0.51}$$

(1)

Where $N$ is the uncorrected SPT blow count and $V_s$ is the shear wave velocity (m/s).

6.2. Mapping of $V_{s30}$ Values

The weighted average value of shear wave velocity $V_{s30}$ in the upper 30 m of ground is denoted as $V_{s30}$ and used worldwide e.g. [53-56], for characterizing a site in terms of the expected characteristics of earthquake shaking, despite
the criticism which is given by several investigators. The classification of soils, for characterizing a seismic site and mainly classifying soils, is based on the average shear-wave velocity in the first 30 m of subsoil [57-58]. This parameter is classically called $V_{s30}$, which is calculated according to the following expression

$$V_{s30} = \frac{30}{\sum_{i=1}^{N} d_i/v_i}$$  \hspace{1cm} (2)$$

Where $d_i$ the thickness of the $i^{th}$ soil layer in meters; $v_i$ shear wave velocity for the $i^{th}$ layer in m.s$^{-1}$ and $N$ is the number of layers in the top 30 m soil strata which will be considered in evaluating $V_{s30}$ values.

A classification system of the site based on $V_{s30}$ values was proposed by national program of research on earthquake risk (NEHRP) also classification of International Building Code (IBC), [62]. So to understand the velocity distribution shear waves average around the region of Mohammadia, average velocity was calculated using the Equation 2 for the location of each drilling. However, the average $V_{s30}$ was calculated to a depth of 30 m.

The Figure 6 represents the $V_{s30}$ map of the “Mohammadia” city was identified. The majority of the study area has a velocity range of 360 to 658 (m.s$^{-1}$). The range of $V_{s30}$ values, specified for each site class, is different in the three methods NEHRP 2003, Eurocode 8 and RPA 2003. The urban city of “Mohammadia” is classified in the category class C for the NEHRP method (2003), Class B for Eurocode 8 and finally class S2 for RPA2003. The site will be considered as a solid one, consisting of sands deposits, very dense gravel and/or strengthened clay from 10 m deep.

![Figure 6. Spatial variation of $V_{s30}$ in the Mohammadia area](image)

7. The Input Earthquake and the Site Response Analyses

In order to obtain the site seismic response results, geotechnical cored test in the town of ”Mohammadia” were realized. Some parameters such as the physical soil characteristic, soil type, thickness, and unit weights are obtained. The analyses are conducted by the one-dimensional linear equivalent method.

The linear approach is the simplest approach to evaluate the ground response. It relies on the principle of superposition. This approach is only suitable for analysis of linear systems. However, the nonlinear behavior of the soil can be approximated by iterative procedure with equivalent linear soil properties. In the equivalent linear analysis, it is required to first estimate the level of shaking in each layer assuming some constant initial values of shear modulus and damping. Later in successive iterations, the shear modulus and damping of the soils that corresponds to the estimated levels of shaking (shear strains) are used in the analysis [65]. Thus, the method considers the nonlinear stress-strain response of the soils indirectly. It is worth remembering that even though it considers equivalent shear modulus and damping where these values are constant throughout the shaking (analysis) in any iteration. A widely used computer program, SHAKE [66] and DYNEQ [67] implemented this equivalent linear approach.

According to the thicknesses and properties of soil layers and by using the seismic response calculation software, designed to soil profiles which are horizontally stratified, i.e. SHAKE91, we calculate for the soil column. This soil profile is modeled, according to the SHAKE91 program, as a multi-degree system of unidirectional freedom. This software is able to calculate the free-field seismic response in terms of acceleration from the excitation that is applied in
the rocky substratum, the amplification function between the rocky substratum and the free surface of the ground, the vibration frequency for this amplification, the response spectrum in pseudo acceleration, relative pseudo speed and relative displacement [22]. By taking the following assumptions into consideration:

The soil column is defined as an insulated soil profile that is known by horizontal layers, surmounting an elastic half-space. Each soil layer is known by its thickness $h_i$, its density $\rho_i$, its damping coefficient $\zeta_i$ and by the shear wave’s velocity $V_s$ or its shear modulus $G_i$. The profile seismic response is generated by the vertical propagation of shearing waves. The excitation wave is introduced as an accelerogram.

The nonlinear behavior (variation of the shearing module and the damping coefficient) is taken into account through an iterative diagram by supposing an equivalent linear behavior [68].

A transfer function is used as a technique for 1D ground response analysis. The seismic signal at the bedrock (input) motion is in the frequency domain obtained by using Fourier transform. Each term in the Fourier series is subsequently multiplied by the Transfer function. The surface (output) motion is then expressed in the time domain using the inverse Fourier transform. The surface ground response is only considered as the input signal product, through the transfer function. Therefore, we obtain a frequency function taking into account the soil movement in the free surface; this is the Fourier transform (FT) of the output signal. We collect, through the inverse Fourier transform (IFT), the temporal ground response at the surface.

In the Mohammadia microzonation case study, 7 scaled acceleration time histories of earthquake of May 21st, 2003 in "Boumerdès" probable magnitude range (Mw = 6.8) were used as input motion for site response analyses by Shake 91 and the average of the acceleration response spectra on the ground surface were determined to obtain the necessary parameters for microzonation. Selected time histories scaled with acceleration values at engineering bedrock level are listed in (As shown in Table 1) and (Figure 7).

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Station</th>
<th>Magnitude</th>
<th>PGA (g)</th>
<th>Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Dar Biada</td>
<td>6.8</td>
<td>0.3169</td>
<td>7.28</td>
</tr>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Ain Defla</td>
<td>6.8</td>
<td>0.0289</td>
<td>5.86</td>
</tr>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Blida</td>
<td>6.8</td>
<td>0.0468</td>
<td>15.78</td>
</tr>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Hussein Dey</td>
<td>6.8</td>
<td>0.2716</td>
<td>6.88</td>
</tr>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Tizi Ouzou</td>
<td>6.8</td>
<td>0.1924</td>
<td>6.24</td>
</tr>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Kaddara (1)</td>
<td>6.8</td>
<td>0.2556</td>
<td>7.48</td>
</tr>
<tr>
<td>Boumerdès 21/05/2003</td>
<td>Kaddara (2)</td>
<td>6.8</td>
<td>0.316</td>
<td>7.86</td>
</tr>
</tbody>
</table>

8. Results and Discussion

In the final results, the microzonation map of the 52 boreholes located in Mohammadia city has been done, taking into account the ground shaking. The peak ground surface acceleration is obtained as a result of seismic ground analysis has been obtained using SHAKE91.

8.1. Site Response Analysis

The Dynamic site response analyses are employed on various cells to question the soil behavior under seismic action. The microzonation maps prepared indicate a medium variation of the amplification factor, the period of the soil column, and the peak spectral acceleration.

Results of site response at the surface made by 1-D linear equivalent codes in correspondence to the conventional bedrock of 52 boreholes have been collected for creating some ground shaking maps for the urban area of Mohammadia city using a geographical information system tool (GIS). The rock motion obtained from ground motion model is assigned at the bedrock level as input in SHAKE91 and evaluated peak acceleration values and acceleration time histories at the top of each sub layer.

The acceleration time history at the surface is obtained as well as the response spectrum acceleration as shown in Figure 8. The PGA at the surface of the selected site in Mohammadia city is amplified due to the effect of the soil profile to be equal to (0.3169 g, 0.0296 g, 0.0468 g, 0.2716 g, 0.1924 g, 0.2556 g, 0.316 g) and the maximum spectral acceleration is varying from 0.51g to 0.96g. This amplification is influenced by both the effect of the input signal and soil profile. The seismic signal placed on the substratum crosses the different layers, and by the mechanism of reflexion/refraction, this signal arrives at the surface after having undergone transformations due to the nature of the ground. Then, the soil profile affects the seismic signal and modifies its characteristics.
Figure 7. Acceleration time histories used as input in the site response analysis of earthquake of May 21st, 2003 in "Boumerdes" (a) Dar Biada Station, (b) Ain Defla Station, (c) Blida Station, (d) Hussein Dey Station, (e) Tizi Ouzou Station, (f) Kaddara (1) Station, (g) Kaddara (2) Station

Figure 8. Contour maps with respect to average spectral accelerations $S_a (g)$ calculated by site response analyses in the city of Mohammadia

Figure 9. Contour map of computed average amplification in the city of Mohammadia
8.2. Amplification Factor Map

The term “Amplification Factor” is hence used here to refer to the ratio of the peak horizontal acceleration at the ground surface to the peak horizontal acceleration at the bedrock. The distribution of average amplification values is also prepared to be considered in the final seismic microzonation map in order to account for site response effects in the study area of Mohammadia city. The classification ranges and the average amplification factor map are presented in (Figure 9). With the average amplification factors varying from 1.74 to 5.51, it can be observed that the average amplification factor for most of the northern part of the study area is in the range of 2.49 - 3.25. This can be considered as significant zone of amplification. The region is moderately amplifying.

9. Conclusion

Seismic microzonation is a technique that takes into account the effects of site (geology, topography, landslide, etc.) on the local seismic hazard around the city of Mohammadia in order to mitigate the risk of the earthquake. In first, we have used the Vs30 estimated values to build a map for his spatial variation on the Mohammadia city that will permit us to propose a classification of the site based in accordance with the NEHRP classification. The nonlinear behavior of soil is modeled by an equivalent linear model which is implemented in SHAKE91. The site amplification is evaluated by assuming the soil column as a dynamic system of multi-degrees freedom in which the input signal is placed at the bedrock and a free field signal will be estimated. From these two signals the site amplification can easily evaluated and used to build the microzonation map. The resulting microzonation maps of seismic site class and spectral ground surface acceleration will be useful to structural engineers to design structures that are relatively resistant to earthquakes using the map information. The lithological site effects observed in the Mohammadia city are very particular and deserve further analysis by carrying out 1D simulation and better characterization of the mechanical properties of soils (in Geotechnical terms in particular). This would make it possible, in particular, to refine the damage scenarios taking in to account, like, the resonant frequencies of the soils observed in the zone and the amplification effects of the seismic movement. The results of the proposed analysis can be considered as an indispensable technical tool that characterizes the seismicity of the city of Mohammadia.

10. References


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Performance Assessment of Screw Piles Embedded in Soft Clay

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Abstract

Screw piles are widely used in a variety engineering applications supplying stability against compression, overturning, moment, uplift tension, and horizontal loads. Screw pile is a famous solution for support light structures, roads and rail signs which have relatively low-capacity foundation. In this study, the behavior of circular (10) mm solid screw pile models embedded in a bed of soft clay soil covering a layer of sandy soil has been studied. The 200 mm thick sand layer was compacted in a steel container with a diameter of 300 mm into four sublayers. The sandy soil layer was compacted at a relative density of 70%. The 300 mm thick soft clay soil bed with \( \text{Cu} < 40 \text{ kPa} \) was compacted in six sub-layers on the sandy bottom layer. Model tests are carried out with screw piles with a length of 300 mm, 350 mm and 400 mm and a helix diameter of 30 mm. Also, single and double helix and different \( \text{S/Dh} \) ratio were used for these piles and a comparative study between screw piles and ordinary piles (without helices) is accomplished. This study revealed that introducing screw pile of double helix increases its bearing capacity in soft clay soil by up to (4-8)\% as compared to a single helix screw pile. The results showed that the behavior of screw pile essentially depends on the geometric properties of the pile. According to the achievements, compressive load capacity of screw piles depends on embedded length, spacing ratio (\( \text{S/Dh} \)) and number of helical plates.

Keywords: Screw Pile; Soft Clay; Cylindrical Shear; Compressive Axial Loading; Helix Plate.

1. Introduction

Soft clay soils are recent alluvial deposits, probably formed over the past 10,000 years described by their flat, featureless surface, Brand and Brenner (1981), British Standard (1986) [1, 2]. These soils are identified by their high compressibility (Cc between 0.19 and 0.44) and their low undrained shear strength (Cu<40 kPa), British Standard (1986) [2]. In general, soft clay soils are stiff when dry and lose this property when become wet. Leakage of sewer lines, floods, rains and lack of evaporation due to buildings or pavements are the popular reasons of increasing moisture content in clayey soils [3]. The soils which have such characteristics cause several problems to geotechnical engineering associated with low bearing capacity, settlements and stability problems.

Pile foundations are the Structure's part that carried and transferred the superstructure load to the bearing ground at a certain depth below the ground surface. Piles are long and thin element that transfer the load through weak, compressible layers or water into deeper soil or rock of high bearing capacity and less compressibility to avoid shallow soil of low bearing capacity [4]. Pile foundations in more conventional civil engineering applications have a wide range of types and sizes and materials are used in practice. A lot of research has been done to find the appropriate type of piles for different geotechnical and structural conditions. First, the shape of the pile was a simple shaft, then it evolved over...
time to adopt complex shapes similar to those that are now used: Franki piles, Omega piles, Fundex piles, drilled piles (CFA), Atlas piles, screw piles and etc. [5]. With construction design challenges and ever-increasing demands for sustainable practices and cost-saving solutions, the construction industry is looking for foundations that offer efficient construction techniques, innovative pile configurations, and novel materials applications.

Owing to their many construction advantages, screw piles are gaining in popularity, especially in projects requiring quick installation and loading of the foundation [6]. Now a day, screw pile foundations have become popular in many countries. The use of screw piles as a deep foundation option has increased significantly in recent years to support various loads, from small loads for multiple applications such as residential housing, solar farms, utilities and renovation projects loads for many applications such as commercial lines, power transmission lines, oil installations and industrial applications [7]. Screw piles are called helical piles or helical anchors, or are structural and deep foundation members used to provide stability to compressive, tension and lateral loadings [8]. Screw piles consist of a steel shaft, either a solid square shaft or a circular tube with one or more helix attached to it [9]. The screw pile design implies the choice of its shaft length and the diameter and a specific arrangement of helices including their number, their diameter (Dh), the ratio of helix spacing over helix diameter (S/Dh) and the embedment depth to helix diameter ratio (L/Dh) of the upper helix. All of these parameters can be influenced on the ultimate capacity of screw pile [10]. On account of the relatively simple installation process compared to traditional deep foundations (e.g. drilling and bored piles) and increasing acceptance in the geotechnical industry, the popularity of screw piles has risen sharply in recent decades. The screw pile like any deep foundation, must embed and transfer load through the active area to a stable ground below. The active area is defined as the area or Depth of seasonal moisture restoration, sometimes called wetting depth. It is the depth or area where the forces of expansion or shrinkage of the soil have an adverse effect on the depth [11]. Screw piles differ from conventional piles in that they are usually made of high strength steel consisting of helices fixed to the shaft at spaced intervals and having a pointed tip to allow for better installation in the ground [12]. There are various dimensions of screw piles that are specific to certain conditions, among which shaft and helical plate diameters, helix pitches, spacing between helical plates and embedment depths are differential points. The screw piles were initially used primarily as anchors, and therefore focused on tensile loads such as transmission tower sand buried pipelines. However, their use has been extended to structures that are subject to compressive, tension and lateral loading [13].

The screw pile system is not suitable for the foundation in gravely or stiff soil, as the helix plates can be damaged during the process of pile installation. Failure due to axial loading of the screw piles can occur either in individual plate bearing or cylindrical shear model. The type of failure can therefore affect the behavior of the piles and their capacity [14]. Screw piles have become the focus of experimental studies in Geotechnical laboratories are studying the resistance to compressive or pull-out forces and the behavior of these piles under axial loads from the structure itself. Screw piles also have good tensile and compressive capacity (2 to 3 times the capacity of a conventional pile [15] and can be used for a variety of soil layers above and below water.

Soft clay soil covers vast areas of the central and southern governorates of Iraq. Soil profile in these areas consist from large depth of soft clay overlaying sandy soil. In such soils, there are substantial difficulties associated with geotechnical design and the implementation of civil engineering structures, difficulties and problems require the use of efficient, efficient and inexpensive solutions, as well as the old buildings are damaged due to the recent earthquakes, And the reconstruction that will be accepted by the country was to be thought of solutions meet all these requirements, which include the use of screw piles, which are characterized by fast and easy installation and multiple uses and low cost and high efficiency.

This paper presents the behavior of screw piles in soft clay soils has focused predominantly on the behavior of multi-helix screw piles loaded in axial compression with varying embedment depth, , number of helix plate, helical plate spacing ratio, S/Dh, and pile length, L, as defined in Figure 1.
2. Materials

2.1. Soft Clay Soil

The soil used in this study was brought from Baquba Brick Factory Workers village within Diyala governorate, Iraq from a depth of two meters from natural ground level. Before the soil preparation stage, trial tests were conducted to control the efficiency of the preparation method. Control tests were performed to determine the variation in shear strength at different water contents (or at different liquidity indices) several trails were made and typical results are showed in Figure 2. The laboratory test results are presented in Table 1. According to the ASTM - standard classification for soil, the soil is classified as (CL).

![Figure 2. Variation of undrained shear strength versus water content for the remolded clay after 48 hrs.](image)

<table>
<thead>
<tr>
<th>Item</th>
<th>Property</th>
<th>Value</th>
<th>Specification</th>
</tr>
</thead>
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<tr>
<td>1</td>
<td>Specific gravity (Gs)</td>
<td>2.84</td>
<td>ASTM D 854 - 2</td>
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<tr>
<td>2</td>
<td>Liquid limit (L.L)%</td>
<td>38.5</td>
<td>ASTM D 4318 - 00</td>
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<td>Plastic limit (P.L)%</td>
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<td>ASTM D 4318 - 00</td>
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<td>4</td>
<td>Plasticity Index (I.P)%</td>
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<td></td>
</tr>
<tr>
<td>5</td>
<td>Clay %</td>
<td>51</td>
<td>ASTM D 422</td>
</tr>
<tr>
<td>6</td>
<td>Silt%</td>
<td>45.7</td>
<td>ASTM D 422</td>
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<tr>
<td>7</td>
<td>Sand%</td>
<td>3.3</td>
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<td>8</td>
<td>Unified Soil Classification System (USCS)</td>
<td>CL</td>
<td>ASTM D 422</td>
</tr>
<tr>
<td>9</td>
<td>Maximum Unit Weight (kN/m³)</td>
<td>17.8</td>
<td>(ASTM D-1557)</td>
</tr>
<tr>
<td>10</td>
<td>Optimum Moisture Content (O.M.C)%</td>
<td>18</td>
<td>ASTM D-1557</td>
</tr>
</tbody>
</table>
2.2. Sandy Soil

This soil is used under soft clay soil, which serves as a stable zone. Fine clean sand from the Karbala Governorate site south of the city of Baghdad in Iraq. Before the testing stage, the sandy soil is dried in the laboratory for 24 hours at 105°C in a drying oven. Then sieved on No.40 sieve to remove the coarse particles. Laboratory tests were carried out on the sandy soil to determine the physical and mechanical properties. The laboratory test results are shown in Table 2. Here should be mentioned; the direct shear test was carried out at a relative density of 70%, which corresponds to (16) kN/m³ dry unit weight. According to the ASTM standard classification for soil, the soil is classified as poorly graded sand (SP).

<table>
<thead>
<tr>
<th>Item</th>
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<th>Value</th>
<th>Specification</th>
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</thead>
<tbody>
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<td>1</td>
<td>Coefficient of Uniformity (Cu)</td>
<td>2.6</td>
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<td>2</td>
<td>Coefficient of Curvature (Cc)</td>
<td>0.84</td>
<td>(ASTM D-422) and ASTM D 2487 (2006)</td>
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<td>SP</td>
<td>(ASTM D-422) and ASTM D 2487 (2006)</td>
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<td>4</td>
<td>Specific Gravity (Gas)</td>
<td>2.63</td>
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<td>5</td>
<td>Cohesion (kN/m²)</td>
<td>0</td>
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<td>6</td>
<td>Angle of Internal Friction(°)</td>
<td>35.79</td>
<td>ASTM D3040-04(2006)</td>
</tr>
</tbody>
</table>

2.3. Model Piles

Twelve steel screw piles with a length of 300, 350 and 400 mm and a circular solid section with a diameter of 10 mm were made of high-strength steel. The diameter of helix plate (Dₜ) are used 30mm with thickness 2 mm and a helical plate pitch (P) of 10 mm. The helix plate were manufactured from steel and welded firmly and accurately to the pile shaft. Two spacing are used 30 mm (S=Dₜ) and 50 mm (S=1.6Dₜ). Figure 4 shows screw pile geometry. The termination of the shaft was a 45% to aid keying during installation. The experimental program is carried out on single pile with different length, helical plate spacing and number of helical plate.
2.4. Soil Container

In general, the soil steel container (test tank) dimensions are determined based on the effectively stressed zone of soil mass from the foundation edge [16]. The steel soil container was manufactured using a 4 mm thick plate having an inner diameter of 30 cm and a height of 55 cm. The base of the container is carried by four small wheels. The containers were painted with two layers of anti-rust paint and two layers of conductive base to withstand corrosion during the test period. Table 2 shows the container used in test models.

![Soil container](image)

**Figure 5. Soil container used in this study**

3. Testing Procedure

Figure 6 shows the experimental program proposed for this study. The tests are conducted in the laboratory of civil engineering of the faculty of Engineering in Diyala University. After testing and soil preparation, self-design laboratory models, shown in Figures 8 and 10 are using respectively to process of pile installation and testing.

![Testing program](image)

**Figure 6. Flow chart of testing program**

3.1. Soil Preparation

This stage begins after completions of the laboratory test required for the used soil in this study.

- Sandy Soil: The required amount of oven-dried natural sand soil passing through No. 40 sieve was prepared at a dry weight of 16 kN/m^3, which corresponds to the dense state. Compaction is manually performed using a steel plate hammer 9.87 kg and of (150) mm diameter.
Soft Clay Soil: The soil bed was prepared at water content of 30% corresponding to cu=30kPa. Weight of 30 kg of natural soil was mixed with enough quantity of water to get the desired consistency. The operation of mixing was performed using electrical mixing; each 15 kg of dry soil was mixed separately till completing the whole quantity, as shown in Figure 7.

After being thoroughly mixed, the soil was placed in six layers in a steel container; each layer was gently levelled with a wooden tamper and then the levelled layer was gently tamped with a manufactured steel plate hammer having a diameter of 9.87 kg and a diameter of (150) mm to remove any trapped air. This process continues for the six layers until reaching 500 mm of soil thickness in the steel container. After the final layer was finished, the top surface was levelled to obtain a flat surface as possible and then covered with polyethylene sheet for two days to achieve a uniform moisture content and prevent moisture loss.

3.2. Pile Installation

The process of installing screw piles is considered to have an influence on the design. Installation in soils that are not detrimental to screw piles will generally not affect the design if standard installation procedures are followed [17]. The installation process conducted after the soil preparation. The model piles were screwed slowly into the ground by applying a torque with appropriate downward force.

The torque applied through the use of a hydraulic torque motor provided the rotational and vertical forces required to install screw pile in the centre of surface of soft clay soil bed to the depth required. The vertical speed and the number of revolutions per minute (rpm) depend on the pitch (P) of the screw pile [18]. The installation of a screw pile must take place in such a way that the screw pile penetrates the ground in an amount equal to the pitch of the helix (P) for each complete revolution in order to minimize the disturbance of the ground [19]. Thus, a controlled displacement installation with a penetration rate of 10 mm/min and a rotational speed of 3.60 rpm was used for all tests. Figure 8 shows that the hydraulic torque motor has been used in the driven screw pile.
3.3. Pile Load Test

In loading cases, the axial compression load is applied to a screw pile through a 3 ton (S-shaped) load cell. Using timer with a constant load penetration rate of 0.5 mm/min in the full test program based on ASTM (2018) standard D1143 for axial compression testing, which are concluded that the range of penetration rate (0.25 to 1.25 mm)/min for cohesive soils). The test is in progress recorded a continuous displacement of the single screw piles until of (12 mm) embedded depth. Figure 9 shown the loading versus time for double helix screw pile with 400 mm length, Dh=30 mm and s=30mm. The load is measured by a digital weighing indicator linked to the load cell. The central settlement of the screw piles cap is read by two digital dial gauge with 0.001 mm sensitivity. Figure 10 shows the pile test device.

![Figure 9. The load-time chart during the process of pile testing](image)

![Figure 10. The device used in testing models](image)

4. Results and Discussions

As the literature indicates, there are several failure criteria methods that predict the ultimate load bearing capacity of screw piles from a load test. Some of these methods are explained by Sakr (2011) [20]. As the Davisson criterion, Brinch Hansen criterion, L1–L2 method, Federal Highway Administration (FHWA) method (5% of the diameter of helix) and ISSMFE (10% of the diameter of helix). The last method, called ISSMFE, is to find the load at a displacement level of 10% of the helix plate.
diameter [21]. The last one was taken over in this investigation.

The behaviour of the screw pile can be better evaluated using the results obtained during compression load tests. Model tests are carried out on ordinary piles (steel pile without helix plates) and have the same circular cross section with diameter (10) mm of screw piles to compare the degree of efficiency with screw piles. Three different L/D ratio were used 30, 35 and 40, also single and double helix with different spacing between helix plates (s), 3cm and 5cm were used. Figure 11 shows the effect of the length of screw piles on its capacity. The increase of the L/D ratio for ordinary pile (steel pile without helix) and screw pile increases the compressive capacity, which is due to the anchoring of long piles in the deep soil layer. In general, the ultimate pile capacity increased with increasing its length, that’s agree with Hamdy (2013) [22] who found that the compression load of screw pile carrying capacities increases with increasing the embedment ratio (L/D) from laboratory tests. In other words, for all piles, the application of the proposed failure criterion on the results led to conclusion that piles installed in soft clay over sandy soil had the greatest ultimate capacity regardless of other factors of pile geometry such as the number of helix and the spacing between the helix plates.

Figure 12 shows the variation of ultimate compressive load of ordinary piles and screw piles in soft clay soil embedded to sandy soil layer. It is clear from Figure 12 is that the ultimate compression force increased with increasing length of embedment of screw piles in sandy soil layer, number of helix and ratio of spacing between helix to helix diameter (S/Dh). This is may be due to the effect of riveting action of screw piles and shear resistance mobilized along the circumference cylindrical and bearing of screw piles and soil.

In general, the screw piles of single and double helix plates have compression force (3.7-8) times more than ordinary pile (without helix plate ). The increase in compression forces are (41, 40, 38, 25) times when ratio of Ls/H change from (0) to (0.33) for ordinary piles, single helix, double helix(S/Dh=1) and double helix (S/Dh=1.6) respectively. In other words, for all piles, the compression capacity of screw pile is strongly affected by the number of helix plate, and that’s agree with Khazaei, Javad, and Abolfazl Eslami (2016) [14] who conclude that adding helix diameter will be very effective in improving the screw pile capacity.

According to the results, in the case of screw pile with double helices, the compression capacity mainly depends on spacing ratio S/Dh. In the current study screw piles by different spacing ratios were tested. 1 and 1.33 S/Dh ratios were used. Figure 13 depicts the relation between spacing ratio (S/Dh) and ultimate compressive load for screw pile of double helix plates at different slenderness ratios. As mentioned above, the ultimate compressive force increases with increase slenderness ratio (L/D). Screw pile of spacing to helix diameter (S/Dh=1) gave (1.4-3.4) times more than that of (S/Dh=1.6) except at L/D=30 which gave approximately the same value. This may be attributed to the contact surface area between helix plates. This area is large at (S/Dh=1) which gave high value of resistance to compression force. In other words, the compressive capacity of screw pile in dense sand decrease when S/Dh ratio increases from 1 to 1.33, also can be observed that the screw pile with single and double helix have a similar behaviour and have better value of compressive capacity in sand than soft clay and the reason could be higher resistance between helices plate and sand.

Figure 11. Variation of ultimate compressive load of screw pile with L/D ratio for different lengths and helix diameters
The presence of helix plates in stable zone (sandy soil) leads to increase compression resistance of screw piles. This behaviour may be attributed to increase in surface area of part of screw pile embedded in sandy soil that cause increase in anchorage resistance.

It was noticed that the deeper screw pile with higher L/D ratios showed greater compression capacity than the shallower piles. Furthermore, screw pile showed more resistance to the applied compression than ordinary piles because of the presence of the helix plates which provides additional anchorage in deep soil layers. The compression force increase with increasing diameter of helices and number of helix plates.

The failure mode of screw piles is examined by cutting the soft clay soil and sandy soil after finish the test of screw pile under...
the compressive load as shown in Figure 14.

The results showed that the type of failure for screw pile with two helix is cylindrical shear surface which occurred in the region between two helix plates while individual helix failure occurred at the base of screw piles which has one helix.

![Single helix plate](image1) ![Double helix plate](image2)

**Figure 14.** Failure mode type of single and double helix plate screw piles

5. Conclusions

The current experimental study can be summarized the following points concluded.

- Quick and easy installation, immediate use and other advantages over the conventional pile system have expanded the use of screw piles as a deep foundation for diverse structures.
- The ultimate compressive capacity of screw piles increase with the increasing number of helices.
- The ultimate compressive capacity of screw piles increase with the increasing the depth of embedment in sandy soil.
- The most effective spacing ratio “S/Dh” is found to equal to 1.
- Laboratory results show that the ultimate compression capacity of screw piles is (4–8) times higher than that of ordinary piles depending upon the number of helices.
- The ultimate compressive capacity of screw piles of S/Dh=1.6 less than for screw pile of S/Dh =1.

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Improving Equipment Reliability and System Maintenance and Repair Efficiency

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Abstract

Mean time to failure of modern machinery and equipment, their individual parts and components can be calculated over the years. Methods for determining the optimal frequency of maintenance and repair, based on the collection and processing of information about the reliability of industrial facilities, during their testing in laboratories and at special sites, as well as through long, operational tests require considerable time and become expensive. The purpose of this work is to develop methods for processing information about the reliability of equipment in automated systems for maintenance and repair, which will reduce the time to collect information on equipment failures and improve the cost-effectiveness of maintenance and repair. Small, multiple-censored right-side samples of equipment operating time for failure are formed as a result of failure data collection in an automated system for equipment maintenance and repair. Calculation of reliability indicators for such samples is performed using the maximum likelihood estimation method. The article presents experimental studies of the accuracy of the maximum likelihood estimates of the parameter of the exponential distribution law for small, multiple right-censored samples. The studies were carried out by computer modeling of censored samples, similar to samples that are formed when monitoring equipment during operation. Methods of simulation modeling of random processes on a computer and methods of regression analysis were used. Analysis show that most of the maximum likelihood estimates obtained from small, multiple-censored right-side samples have significant deviations from the true values. A technique for improving the accuracy of maximum likelihood estimates is proposed. The scientific novelty is regression models are constructed that establish the relationship between the deviation of the maximum likelihood estimate from the true value and the parameters characterizing the sample structure. These models calculate and introduce corrections to maximum likelihood estimates. The use of the developed regression models will reduce the time to collect information about the reliability of the equipment, while maintaining the reliability of the results.

Keywords: System Maintenance and Repair; Equipment Reliability; Censored Samples; Maximum Likelihood Method; Computer Simulation.

1. Introduction

The essence of the system maintenance and repair lies in the fact that after a certain amount of time worked, various types of repair work are carried out aimed at restoring the equipment (maintenance, current or capital repairs). The system of maintenance and repair is a regulatory information base necessary for the development and scheduling of maintenance and repair. It contains the scope of work for maintenance, current or capital repairs; structure and frequency of maintenance and repair; norms of labor intensity and repair time; consumption rates of basic materials, components, spare parts; reserve standards. Regulatory and information base is formed on the basis of industry average indicators. As
a result, uniform standards and rules for maintenance and repair are applied at enterprises that differ: in the climatic conditions in which the industrial equipment operates, its service life and its degree of wear, type and performance, and qualifications of the service personnel. Therefore, these rules may not be optimal for this particular enterprise.

The lack of a connection between the operating conditions of the equipment in the enterprise and the regulatory base of the system leads to the fact that the current the system maintenance and repair hampers the development and introduction of new technologies in repair. Improving repairs, introducing technical diagnostics methods to periodically monitor the condition of the equipment increases the operational reliability of the equipment. However, the frequency of planned work, which is determined by current regulations, does not change. As a result, the costs associated with the introduction of new developments do not pay off.

In some studies such as Chin-Chih (2014), Kizim (2016), Baskakova et al. (2016) and Knopik and Klaudiusz (2019), the authors concluded that the principles of strict regulation of the structure and duration of the repair cycle have a negative impact on the efficiency of the maintenance and repair system [1-4]. It is noted that the duration of the repair cycle must be different even within the framework of one model of tools, machines, since the service life of the elements of various tools, machines is different. Also, the stochastic process of wear and difference between the lifetimes of tools and machines represents additional conditions that are not taken into account by the system. Planning and calculation of indicators and standards based on repair units cannot take into account the individual characteristics of the machines, operating conditions and varying degrees of depreciation, the degree of technical preparation for production, and the technological level. The existing branch of maintenance and repair is based on this basis. Repair units are a specific type of equipment. To develop standards for the system maintenance and repair of individual elements of electrical equipment, it is necessary to perform an analysis of electrical equipment failures when collecting information about reliability. On the scale of the entire industry, it is practically impossible to carry out this work due to the high material costs and the difficulty of obtaining reliable information.

Liu et al. (2019), it was recommended to adjust the system maintenance and repair, developed in the design of machines, during operation. Adjustment is necessary because of the random nature of the operation of machines and the random change in their technical condition, accumulated experience in the operation of machines [5]. With each frequency adjustment, the system maintenance and repair perform the following major tasks:

- Assessment of reliability indicators and development of a list of changes to the current the system maintenance and repair;
- Development of recommendations for improving the methods of the system maintenance and repair;
- Verification of the results of the adjustment of the system maintenance and repair on a limited number of machine samples by conducting their trial operation;
- Final development of the maintenance system for all machines in operation and its implementation.

Most of all, this principle is implemented in the USA, where, as noted by Jianming et al. (2019), Ditho et al. (2018), Shojaei and Volgers (2017) and Yan-qing et al. (2014), there is a complete lack of a standard approach in choosing strategies and tactics, forms of maintenance and repair in each specific case, since it is considered that each enterprise is unique in its own way and there are no two similar [6-9].

These deficiencies can be eliminated by creating and implementing the system maintenance and repair at enterprises, which links the frequency of the system maintenance and repair with the operational reliability and technical condition of electrical equipment.

Adaptability of the system maintenance and repair is the property that can be described as the ability of the system to self-adjust, adapt to changes in the factors characterizing the operation process, equipment operating conditions, and changes in the reliability of electrical equipment. The adaptability of the system maintenance and repair is provided by changing the schedule of maintenance and repair in accordance with changes in the reliability of the equipment of the enterprise or its structural unit (shop, site). This is achieved by the fact that the period between the various planned work of the system maintenance and repair τ is calculated on the basis of mathematical models of maintenance, repair and overhaul, taking into account the distribution of time between failures, average recovery time and other indicators of the reliability of electrical equipment. The system maintenance and repair schedule is constantly updated as information on equipment reliability is accumulated and updated.

According to the modern level of development of science and technology, an automated system for maintenance and repair should perform the following functions:

- Accumulation of statistical information on equipment failures and its individual structural elements. Finding the law of distribution of time between failures of equipment and the calculation of its parameters;
• Calculation of the optimal frequency of maintenance and repair for this type of equipment and its individual structural elements;

• Analysis of equipment failures, allowing for the calculation of the frequency of maintenance, current and capital repairs to take into account only those failures, the causes of which are eliminated as a result of the corresponding type of maintenance and repair;

• The creation and maintenance of a regulatory information base for maintenance and repair for various types of electrical equipment and some of the most frequently denying its elements.

Thus, an automated maintenance and repair system should be an integrated environment designed to accumulate information about equipment failures and its technical condition, optimize the maintenance and repair system, and systematize the experience of maintenance personnel.

The general algorithm of functioning of the adaptive system of maintenance and repair is shown in Fig.1. To determine the mean time between failures and average recovery time, the collection and processing of statistical information about equipment reliability is organized. Then, using mathematical models of maintenance and repair, using the found reliability indicators and the distribution function of add-ons for failure, the frequency of maintenance and repair works is calculated. At the last stage, a maintenance and repair schedule is drawn up. After a certain period of time, after updating the statistical information about the reliability of electrical equipment, the calculation cycle is repeated and a new maintenance and repair schedule is compiled, corresponding to a new level of reliability.

From the algorithm of functioning of the adaptive maintenance and repair system (Figure 1), it can be seen that the main elements of the mathematical support of the system are the methods of processing statistical information about equipment reliability and mathematical models of maintenance and repair, designed to calculate the recovery interval.

Figure 1. Algorithm of functioning of the automated system of maintenance and repair

The Bykhelt and Franken (1988) describes the main strategies for work in the system of maintenance and repair, equipment recovery strategies, and proposed Equation 1-6 for calculating the recovery interval \( \tau \) [10].

**Strategy 1.** If the object has worked without failures for a specified time interval \( \tau \), then preventive maintenance is carried out. Restorations that are made after failures are called abnormal. Both preventive and disaster recovery are complete. The intensity of operating costs \( R(\tau) \) is calculated by the Equation 1:

\[
R(\tau) = \frac{C_a F(\tau) + C_n F'(\tau)}{\int_0^\tau \cdots}
\]

Where \( C_a \): Average disaster recovery costs; \( C_n \): Average preventative recovery costs; \( \tau \): Maintenance period; \( F(t) \): functions of distribution of time between failures.

This is a solution to the equation \( dR(\tau)/d\tau = 0 \) or:

\[
\lambda(t) \int_0^\tau \cdot \cdots = \frac{C}{1-C}
\]
Strategy 2. In case of failure, the object undergoes disaster recovery, which is the minimum recovery. Regardless of the age of the system, preventive maintenance is systematically carried out at fixed points in time, that is, full recovery is carried out.

Intensity of operating costs:
\[ R(t) = \frac{C_n + C_A \Lambda(t)}{\tau} \]  \hspace{1cm} (3)

Where \( \Lambda(t) \) cumulative failure rate:
\[ \Lambda(t) = \int_0^t \lambda(t) dt \]  \hspace{1cm} (4)

The optimal recovery interval is a solution to the Equation 5:
\[ \tau \lambda(\tau) - \Lambda(\tau) = \frac{C_n}{C_a} \]  \hspace{1cm} (5)

2. Materials and Methods

The main method of calculating reliability indicators for censored samples is the maximum likelihood method. The essence of the maximum likelihood method is as follows [12, 13]. The likelihood function is constructed for sampling random variables with a known distribution law. The likelihood function is:
\[ L = A \prod_{i=1}^{n} f(t_i) \prod_{j=1}^{m} (1 - F(\tau_j)) \]  \hspace{1cm} (7)

Where \( A \): Constant rate; \( t_i \): Time to failure of the observed object; \( \tau_j \): Censored operating time; \( n \): Number of failures; \( m \): Number of censored operating time; \( F(t) \): Functions of distribution of time between failures; \( f(t) \): Distribution density.

To find the maximum likelihood estimates of the exponential distribution, it is necessary to solve the Equation 8:
\[ \frac{d \ln L}{d t} = 0 \]  \hspace{1cm} (8)
For the exponential law, the distribution density and the distribution function are respectively equal to:

\[ f(t) = \lambda e^{-\lambda t} \]  
\[ F(t) = 1 - e^{-\lambda t} \]  

(9)  

(10)

The likelihood function is

\[ L = f(t_1) \cdot f(t_2) \cdots \cdot f(t_r) \cdot F(\tau_1) \cdot F(\tau_2) \cdots \cdot F(\tau_n) \]  

(11)

After substitution (9) and (10) in (11), take the logarithm likelihood function

\[ \ln L = r \cdot \ln \lambda - \frac{r}{\lambda} \left( \sum_{i=1}^{r} t_i + \sum_{j=1}^{n} \tau_j \right) \]  

(12)

Where \( \lambda \): exponential distribution parameter; \( t \): operating time to failure; \( r \): operating time to censoring; \( r \): the number of operating time to failure; \( n \): the number of operating time to censoring.

Differentiating Equation 12 by \( \lambda \) and equating to 0, we get:

\[ \frac{\partial \ln L}{\partial \lambda} = r - \frac{r}{\lambda} \left( \sum_{i=1}^{r} t_i + \sum_{j=1}^{n} \tau_j \right) = 0 \]  

(13)

From Equation 13, we obtain the point estimate of the parameter of the exponential distribution:

\[ \lambda = \frac{r}{\sum_{i=1}^{r} t_i + \sum_{j=1}^{n} \tau_j} \]  

(14)

The article presents experimental studies of the accuracy of the maximum likelihood estimate for the parameter of the exponential distribution law for small, multiple right-censored samples. Such samples arise during the observation of failures during operation or when testing according to the plan \([N, U, Z]\).

\([N, U, Z]\) – is a test plan, according to which \(N\) objects are tested simultaneously. The objects that failed during the tests do not restore or replace, each object is tested during the operating prime \(z_i\), where \(z_i = \min(t_i, \tau_i)\), \(t_i\) – is the operating time to failure, \(\tau_i\) is the operating time before \(i\)-th object is removed from the test.

The studies were carried out using computer modeling of censored samples, similar to samples formed in the course of studying equipment reliability indicators in an automated system for maintenance and repair.

To conduct research, an algorithm and a program for simulating the process of computer failures arising during tests according to the plan \([N, U, Z]\) have been developed.

The following algorithm was used to form the sample that was censored to the right several times:

1. A random variable \(t\) is generated, distributed according to the studied exponential distribution law, calculated by the Equation 15;

\[ z = -\frac{1}{\lambda} \ln R \]  

(15)

Where \(R\): random variable uniformly distributed over the interval \((0, 1)\).

2. A random variable \(\tau\) is generated, distributed according to a censoring distribution law. The right-truncated normal distribution law was used as a censoring law.

3. The resulting random variables are compared. If \((t < \tau)\), a random variable \(t\) is added to the simulated sample, which corresponds to the time to failure. If \((t > \tau)\), a random variable \(\tau\) is added to the simulated sample, which corresponds to the operating time before censoring.

4. The modeling process continues until the number of random variables obtained becomes equal to a given number of members of the sample \(N\) (sample size).

The computer simulated samples of random variables that were repeatedly censored on the right, of volume \(N = 5, 10, 15, 20, 25\). The generation of samples was carried out under the following restrictions

\[ 5 \leq N < 10, \quad q \geq 0.5 \]  
\[ 10 \leq N < 20, \quad q \geq 0.3 \]  
\[ 20 \leq N \leq 50, \quad q \geq 0.2 \]  

(16)
Where $q$ – degree of sample censoring.

The number of formed samples $V$ for each value of $N$ is 3000. For each sample, the maximum likelihood method was used to calculate the estimates of the exponential distribution and their relative deviations $\delta$ from the true values using the Equation 17:

$$\delta = \frac{\hat{\lambda} - \lambda_{OMM}}{\lambda}$$

(17)

Where $\lambda$: true value of the exponential distribution parameter; $\lambda_{OMM}$: maximum likelihood estimate of the exponential distribution.

The studies posed the problem of studying the accuracy of maximum likelihood estimates for various samples. Therefore, the parameter $\lambda$ of the studied distribution law was calculated for each generated sample using a random number uniformly distributed over the interval $[0, 1]$ according to the Equation 18:

$$\lambda = 0.6 + 0.4 \times RAND()$$

(18)

Where $RAND()$ is the function of generating a random number uniformly distributed on the interval $[0, 1]$.

According to the simulation results, histograms of relative deviations of the maximum likelihood estimates of the exponential distribution are constructed. The ordinate axis shows the percentage of estimates $p$ of the total number falling into this interval. The results are shown in Figure 2.

![Figure 2. Relative deviations of the maximum likelihood estimate](image)

These experimental data show that most of the maximum likelihood estimates obtained from small, repeatedly censored samples from the right, have significant deviations from the true values. For example, 1% of the estimates of the exponential distribution for $N = 5$ have relative deviations from 10 to 20; 4% - from 5 to 10; 8% - from 3 to 5. With increasing sample size $N$, the accuracy of the estimates increases. When $N = 25$, the relative deviations of the estimates of the exponential distribution law do not exceed 2. Despite this, 2% of the estimates have relative deviations from 1.5 to 2; 3% - from 1 to 1.5; 9% - from 0.75 to 1; 12% - from 0.5 to 0.75. At $N = 5, 10, 15$, a strong shift in the maximum likelihood estimates is clearly visible.

In general, we can conclude that the accuracy of the maximum likelihood method for values of $N < 25$ is low. The relative deviation of estimates from the true values can reach 3 or more, and half of all estimates have deviations greater than 0.3, depending on the sample size.

This article proposes a method for improving the accuracy of maximum likelihood estimates for small, repeatedly censored samples during tests carried out according to the $[N, U, Z]$ plan.

The purpose of the conducted research in general form can be formulated as follows: obtaining mathematical models that establish a relationship between the relative deviation of the maximum likelihood estimates from the true value of the exponential distribution parameter and parameters characterizing the sample structure.

The solution of the task consisted of the following stages:

1. Computer simulation of samples of random variables repeatedly censored to the right, distributed according to the exponential law, according to the algorithm described above.
To avoid repetition of pseudo-random number sequences, a random number was generated based on system time before each sample was formed. To do this, we used the standard \( R\!A\!N\!D(-1) \) function with a negative argument \(-R\!A\!N\!D(-1)\). Obtaining a sufficient difference in the system time, when generating samples, was carried out using a time delay of up to 30 milliseconds before each cycle of forming a repeatedly censored sample.

2. Calculation of sample parameters characterizing its structure. To describe the structure of the formed sample of random variables, we used five standard sampling parameters [14, 15]:

- Degree of censoring
  \[ X_1 = q = \frac{k}{N} \]  
  Where \( k \): number of complete random variables, \( N \): the number of members in the sample.

- The coefficient of variation:
  \[ X_2 = \frac{S}{\bar{Z}} \]  
  Where \( S \): estimation of the standard deviation of all random variables in the sample; \( \bar{Z} \): expected value of all members of the sample.

- Coefficient of variation of total random variables:
  \[ X_3 = \frac{\tilde{S}_\eta}{\bar{Z}} \]  
  Where \( \tilde{S}_\eta \): estimation of the standard deviation of total random variables.

- Empirical skewness coefficient:
  \[ X_4 = \tilde{A} = \frac{Z - \bar{Z}}{\sqrt{\bar{Z} - \bar{Z}^2}} \]  
  \[ X_5 = \tilde{E} = \frac{\tilde{\mu}_4}{S^4} - 3 \]  
  Where \( \tilde{\mu}_4 \): central moment of the fourth order.

Another five parameters are mathematical expressions made up of standard sampling characteristics:

- The ratio of the expectation of total random variables to the expectation of all members of the sample:
  \[ X_6 = \frac{Z_{\bar{Z}}}{\bar{Z}} \]  

- The ratio of the expectation of censored random variables to the expectation of all members of the sample:
  \[ X_7 = \frac{\tilde{Z}_\eta}{\bar{Z}} \]  

- Relative deviation of the expectation from the middle of the variation range:
  \[ X_8 = \frac{R - \bar{Z}}{2\bar{Z}} \]  
  Where \( R = Z_{\text{max}} - Z_{\text{min}} \); variation scale, \( Z_{\text{max}}, Z_{\text{min}} \): respectively, the maximum and minimum value of a random variable.

- Relation of median to expectation of random variables:
  \[ X_9 = \frac{\bar{M}}{\bar{Z}} \]  
  Where \( \bar{M} \) median.

- Mode relation to expected value:
\[ X_{10} = \frac{\bar{M}_0}{Z} \]  

(28)

All parameters are measured in relative units and do not depend on the absolute values of random variables. This has achieved the adequacy of the regression equations obtained in an experiment on a computer with regression equations describing similar dependencies for a set of samples of failures that are formed during the observation of the equipment.

3. Calculation of maximum likelihood estimates of the parameter of the exponential distribution law.

4. Calculation of the dependent parameter - the deviation of the maximum likelihood estimate from the true value by the Equation 29:

\[ \bar{\lambda} = \frac{\lambda_{max}}{\lambda_{true}} \]

(29)

5. Building regression dependencies. As a result of the research, regression mathematical models were constructed that establish a connection between the deviation of the maximum likelihood estimate from the true value and the parameters characterizing the sample structure. For each sample size \(N\), a regression equation is constructed.

3. Results and Discussion

Mathematical models are constructed in the class of linear regression equations of the form:

\[ y(x) = b_0 + b_1 x_1 + \cdots + b_{10} x_{10} \]

(30)

The method of regression analysis is taken according to Young (2018) [16]. The initial data for calculating the coefficients \(b_0, b_1, \ldots, b_{10}\) is the set of parameters \(x_1, x_2, \ldots, x_{10}\) characterizing the sample structure, and the deviations of the maximum likelihood estimate \(y\), represented as a matrix \(X\) and a vector \(Y\):

\[
X = \begin{bmatrix}
  x_1 x_1 x_1 \cdots x_1 x_m \\
  x_2 x_2 x_2 \cdots x_2 x_m \\
  \vdots \\
  x_{10} x_{10} x_{10} \cdots x_{10} x_m 
\end{bmatrix}, \quad Y = \begin{bmatrix}
  y_1 \\
  y_2 \\
  \vdots \\
  y_{10} 
\end{bmatrix}
\]

(31)

A system of normal equations is compiled.

\[
Vb_0 + b_1 \sum_{i=1}^{V} x_1 i + \cdots + b_m \sum_{i=1}^{V} x_m i = \sum_{i=1}^{V} y_i ,
\]

(32)

\[
b_0 + \sum_{i=1}^{V} x_1 i + b_1 \sum_{i=1}^{V} x_1 x_1 i + \cdots + b_m \sum_{i=1}^{V} x_1 x_m i = \sum_{i=1}^{V} x_1 y_1 ,
\]

\[
b_0 \sum_{i=1}^{V} x_m i + b_1 \sum_{i=1}^{V} x_1 x_m i + \cdots + b_m \sum_{i=1}^{V} x_m x_m i = \sum_{i=1}^{V} x_m y_1 .
\]

This system of equations is solved by the inverse matrix method.

The parameters of the regression equation are calculated by the Equation 33:

\[ b_j = \sum_{i=1}^{n} C_{ij} A_{ij} , \quad b_0 = \bar{y} - h_1 x_1 - \cdots - h_m x_m \]

(33)

Where \(C\) is the coefficient matrix for the unknown parameters \(b_0, b_1, \ldots, b_{25}; C^{-1}\) is inverse matrix \(C\); \(C_{ij}\) is the element at the intersection of the \(i\)-th row and the \(j\)-th column of the matrix \(C^{-1}\); \(n\) is the number of equations in the system or rows in the matrix \(C\); \(\bar{y}\) is the average value of the resulting attribute \(y\); \(A_{ij}\) is expression

\[ \sum_{i=1}^{V} x_j y_i - V x_j y \quad (j = 1, 2, \ldots, m) \]

(34)

The residual dispersion estimate \(\sigma_{OPT}^2\) is:

\[ \hat{s}_{\text{res}}^2 = \frac{\sum_{i=1}^{V} (y_i - y(x_i))^2}{V - m - 1} \]

(35)

Where \(y_i\) is the deviation of the maximum likelihood estimate of the parameter of the distribution function, calculated in the simulation as the ratio of the true value of the parameter to its estimate; \(y(x_i)\) is the value of the maximum likelihood
estimate deviation calculated by the regression equation.

The calculation of the statistics $t$ and $F$ to assess the significance of the coefficients of the regression equation and the regression equation as a whole is carried out according to the Equations 36-37:

$$\dot{t}_i = \frac{|b_j|}{s_{bj}}$$ (36)

$$s_{b0} = s_{oct} \sqrt{\frac{1}{n} + \sum_{m} x_j x_k C_{jk}^{-1}}, \quad s_{bj} = s_{oct} \sqrt{C_{jj}^{-1}} \quad j = 0, 1, 2, \ldots, m. \quad \text{and} \quad i = 1, 2, \ldots, n. \quad (37)$$

Where $C_{jj}^{-1}$ is the diagonal element of the inverse matrix.

$$F = \left(\frac{Q_1}{Q_{oct}}\right) \left(k_2/k_1\right)$$ (38)

Where $(Q_1 = Q - Q_{oct})$ is sum of squares characterizing the influence of signs $x$; $Q_{oct}$ is the residual sum of squares characterizing the influence of unaccounted factors; $Q$ is the total sum of squared deviations of the resultant mark; $k = V - m - 1$, $k = m$: degrees of freedom.

$$Q = \sum_{i=1}^{V} (y_i - \bar{y})^2$$ (39)

$$Q_{oct} = \sum_{i=1}^{V} (y_i - y(x_i))^2$$ (40)

Where $Q$: the total sum of squares of the resultant mark; $Q_{np}$: the total sum of squares characterizing the influence of marks, $Q_{oct}$: the residual sum of squares – the influence of unrecorded factors. These equations are significant. The coefficients $b0, b1, \ldots, b10$ and the parameters of the significance of the regression equations are shown in Table 1.

**Table 1. Coefficients and significance parameters of regression equations**

<table>
<thead>
<tr>
<th></th>
<th>b</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>b0</td>
<td>0.1</td>
<td>1.079</td>
</tr>
<tr>
<td>b1</td>
<td>-0.8</td>
<td>-1.483</td>
</tr>
<tr>
<td>b2</td>
<td>0.036</td>
<td>-0.036</td>
</tr>
<tr>
<td>b3</td>
<td>0.017</td>
<td>-0.008</td>
</tr>
<tr>
<td>b4</td>
<td>-0.054</td>
<td>-0.077</td>
</tr>
<tr>
<td>b5</td>
<td>0.019</td>
<td>0.019</td>
</tr>
<tr>
<td>b6</td>
<td>-0.076</td>
<td>0.033</td>
</tr>
<tr>
<td>b7</td>
<td>0.021</td>
<td>0.101</td>
</tr>
<tr>
<td>b8</td>
<td>-0.354</td>
<td>-0.38</td>
</tr>
<tr>
<td>b9</td>
<td>0.006</td>
<td>-0.008</td>
</tr>
<tr>
<td>b10</td>
<td>0.01</td>
<td>-0.008</td>
</tr>
<tr>
<td>Q</td>
<td>91</td>
<td>164</td>
</tr>
<tr>
<td>Q_{np}</td>
<td>56</td>
<td>124</td>
</tr>
<tr>
<td>Q_{oct}</td>
<td>35</td>
<td>40</td>
</tr>
</tbody>
</table>

The resulting regression equations can improve the accuracy of the maximum likelihood estimate by introducing an amendment to the maximum likelihood estimate using the Equation 41:

$$\dot{\lambda}_{KOH} = \dot{\lambda}_{KOH} \cdot \bar{y}(x)$$ (41)

Where $\dot{\lambda}_{KOH}$ is the final estimate of the distribution parameter.

Studies have evaluated the effectiveness of the constructed regression models. Another experiment was conducted to simulate the failure of equipment on a computer. Modeling samples of failures was performed according to the algorithm described above in the article. The evaluation of the effectiveness of the equations obtained was performed using newly generated random samples, and not according to those for which these equations were obtained. This proves the possibility of using the developed models for various types of equipment.
For each simulated sample were used to calculate: estimation of the maximum likelihood of the exponential distribution parameter, the corrections to the maximum likelihood estimate (30) and the final estimate of the exponential distribution parameter using expression (41).

In total, 3000 samples were simulated for each experiment for each number of N sample members. The results of studies of the effectiveness of using the constructed regression equations for the exponential distribution law are shown in Table 2 and in Figures 3 to 7. The abscissa shows the relative deviations of the maximum likelihood estimates from the true value. The ordinate is the number of k estimates with a given relative deviation. The total number of estimates is equal to the number of simulated samples - 3000.

Table 2. The variance of the initial and final relative deviations of the estimates of the exponential distribution

<table>
<thead>
<tr>
<th>N</th>
<th>δ Initial</th>
<th>δ Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.945</td>
<td>0.088</td>
</tr>
<tr>
<td>10</td>
<td>0.318</td>
<td>0.035</td>
</tr>
<tr>
<td>15</td>
<td>0.179</td>
<td>0.023</td>
</tr>
<tr>
<td>20</td>
<td>0.162</td>
<td>0.028</td>
</tr>
<tr>
<td>25</td>
<td>0.125</td>
<td>0.029</td>
</tr>
</tbody>
</table>

Figure 3. Initial and final relative deviations of the estimate $\lambda$ for $N=5$

Figure 4. The initial and final relative deviations of the estimate $\lambda$ for $N = 10$
Figure 5. Initial and final relative deviations of the estimate $\lambda$ for $N=15$

Figure 6. Initial and final relative deviations of the estimate $\lambda$ for $N=20$

Figure 7. The initial and final relative deviations of the estimate $\lambda$ for $N=25$
The use of the developed models significantly improves the accuracy of maximum likelihood estimates. According to Table 2, it can be seen that the variance of the relative deviations of the estimates of the exponential distribution law decreases by a factor of 4–10. At the same time, with a decrease in the number of N sample members, the efficiency of using the developed models increases. It can be seen from Table 2 that when N is reduced from 25 to 10, the variance of the relative deviations of the maximum likelihood estimates δ initial increases by a factor of 2.5 from 0.125 to 0.318, and the variance of the relative deviations of the final estimates δ final obtained from the application of the developed technique increases very slightly from 0.029 to 0.035. In practice, when testing equipment, this will reduce the time of testing while maintaining the reliability of the calculated reliability indicators.

Software tools for the automated maintenance and repair system have been developed, which are designed to calculate the economically optimal frequency of maintenance, current and capital repairs. They automate the calculation of the optimal schedule of maintenance and repair. Their use increases the capabilities of the enterprise in self-optimizing the cost of operating electrical equipment. Software tools use the developed methodology, expand the functions of the existing system of maintenance, and repair. They should be considered and used as an addition to it.

The program allows the user to:

- create and correct reference databases on equipment, types of failures and maintenance costs, current and capital repairs;
- carry out operational input, adjustment and accumulation of information on maintenance, repair and equipment failures;
- calculate the optimal frequency of maintenance, current and capital repairs;
- carry out the output of the calculation results on the screen, printer or file;

The program consists of three modules interconnected with each other: "Directories", "Maintenance and Repair Cards", "Calculations".

The program has five directories. The “Equipment Groups” directory allows you to split all equipment in random order into groups and store the following data: equipment group code, group name. The directory "List of equipment" is designed to create a list of equipment installed at the enterprise. The following data is filled for each piece of equipment: serial number, group, name, release date, start date, start date of observation. In the "Elements of equipment" directory, the user can break each type of equipment into separate elements. Thus, it is possible to calculate the optimal frequency of maintenance and repairs, not only in general for the type of equipment, but also for its individual elements. The following data is entered in the "Equipment components" directory: equipment group, element number. Reference book "List of failures" are entered characteristics of failures: the name of the failure, the group of failure. Information on the costs associated with maintenance, repairs and overhauls, and technical diagnostics is entered in the “Cost of Repair” directory.

The second part of the program "Maintenance and Repair Cards" contains all the necessary means for entering, adjusting and storing operational information about routine maintenance in the maintenance and repair system and equipment failures.

In the third module of the "Calculations" program, calculations of the optimal frequency of maintenance, current and capital repairs, and technical diagnostics are performed.

4. Conclusions

According to the results of the study, we can formulate the following results and conclusions.

- The characteristics of samples of failures generated during operational observations or equipment tests using the \([N, U, Z]\) plan are considered. It is shown that the samples are small, repeatedly censored on the right.

- For the formation of samples of random variables according to the test plan \([N, U, Z]\) on a computer, an algorithm was developed that ensures the adequacy of simulated samples on a computer, samples that are formed as a result of monitoring equipment.

- Experimental studies were conducted to analyze the accuracy of the maximum likelihood estimates of the exponential distribution law over the formed samples of random variables. The experimental results show that the accuracy of the maximum likelihood method for \(N < 25\) is low. The relative deviation of individual maximum likelihood estimates from the true values can reach 5 or more, and half of the estimates have deviations of 0.3, depending on, the degree of censoring, the sample size.

- Sampling parameters are proposed that describe its structure. All of them are measured in relative units and do not depend on the absolute values of random variables.
A computational experiment was performed on a computer to simulate samples of random variables that are adequate to samples of equipment time to failure that are formed during tests conducted according to the \([N, U, Z]\) plan. The parameters characterizing its structure were calculated for each sample. Based on the results obtained in a computational experiment, mathematical models were constructed in the form of regression equations, establishing the relationship between the relative deviation of the estimated parameters of the studied distribution laws obtained by the maximum likelihood method and the sample parameters characterizing its structure.

A methodology has been developed for estimating the parameters of the exponential distribution law, which consists in the fact that the maximum likelihood estimates are corrected by means of corrections. Amendments to the estimates are determined using mathematical models developed in the research process.

The study of the effectiveness of the methodology for estimating the parameters of the exponential distribution law is performed. The results show that it improves the accuracy of maximum likelihood estimates of the exponential distribution law.

5. Conflict of interest
The authors declare no conflict of interest.

6. References


Experimental Investigation on Efficiency Factor of Pile Groups Regarding Distance of Piles

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Abstract

There are a lot of the parameters which affect pile group behavior in soil. One of these factors is the distance of piles from each other. The impact of distance on pile groups in sand has been investigated through some researches, whereas most of them have not represented an exact estimation according to the continuous change of the distance in sand. Moreover, most of previous investigations have considered two piles as a perfect group. Since two-pile group has the least interaction effect among piles, it cannot suitably demonstrate the influence of spacing. In this lecture, several 4-pile groups modeled with different spacing were subjected to axial loading in laboratory. The pile groups were free-head with length to diameter ratio of 13.5. The piles are designed in a way which the shaft resistance of piles can be completely mobilized through the test. Then, the bearing capacities of pile groups are measured and compared with the single pile's resistance in order to calculate the efficiency coefficient of the groups. It is revealed that the distance is noticeably effective in efficiency factor and this effectiveness, non-linearly decreases by increase of spacing. The results show that the efficiency coefficient is changing between almost 1 and 1.4.

Keywords: Pile Group; Spacing; Distance; Efficiency; Axial Loading.

1. Introduction

Since the computational formulas are too conservative, designers take the advantages of practical experiment, satisfied with concepts such as the efficiency coefficient and the ratio of the pile group settlement. These two factors are properly able to represent the performance of the group. Nevertheless, due to the cost of the real size test, the laboratory models are more reasonable.

The influence of distance on group piles' behaviour has been investigated through some researches and it is clear that the increase of piles' spacing declines the effect of piles on each other in most aspects [1, 2], but most of them have not represented an exact estimation according to the continuous change of distance in sand. Moreover, most of previous investigations have considered two piles group as a perfect group. Because the interaction between piles enhances with increasing of the number of piles in group, in this lecture, 4-pile groups are investigated to better show the influence of spacing. In groups with more piles the loads are not distributed equally among piles. Therefore, four piles are suitable.

In one of the previous works, Vesić (1969) declared that the efficiency of the pile group in sandy soil is higher than 1 only if the density of sand or the distance between the piles is not high. Moreover, in the distance ratio of 2 to 3 times of the diameter, the efficiency reaches maximum state (1.3 to 2, respectively) [3]. Poulos and Davis (1980) have obtained...
the interaction factors for the bivalent pile groups in two modes of friction (floatation) and end bearing. These factors, which are used to estimate settlements, display how piles influence each other. The interaction factor decreases with the increase of distance-diameter ratio. As a result, their influence declines [4].

Robinsky and Morrison (1964) investigated the amount of compactness and relative density around the pile using radiographic technique. It was found that in relatively loose sand ($D_r = 17\%$), soil movement proceeds in range of 3 to 4 times the diameter of the pile from the sides and 2.5 to 3.5 times of diameter, below the pile point. In a relatively dense sand ($D_r = 35\%$), the amount of motion is slightly larger and equals 4.5 to 5.5 times the diameter of the pile from the sides and 3 to 4.5 times below the pile point [5]. Ismael (2001) conducted some experiments on sand in real size. He concluded that pile group with distance-diameter ratio of 2 and 3 has the efficiency coefficient of 1.22 and 1.93, respectively [6]. Basile (2003) declared that the two parameters are effective in the behaviour of pile group and load distribution. First: the interaction of adjacent piles. Second: the hardened soil which is enclosed among pile groups [7].

Le Kouby et al. (2016) demonstrated that efficiency coefficient based on the pile spacing ratio, in the tip resistance, increases or stabilizes by values always smaller than one. The efficiency coefficient exhibits a decrease with an increasing pile spacing ratio when considering shaft friction [8].

2. Materials and Testing

The experiments were conducted in the laboratory of the Faculty of Civil Engineering, Tabriz University. In these experiments, 5 different modes were tested. Four of which were pile groups with different spacing, and the fifth one was a single-pile which was used to obtain an efficiency coefficient.

2.1. Properties of Soil

0.4 cubic meters of sieved sand (sieve no.20) used in the experiments. Three experiments were conducted to determine properties of the soil. The soil profile obtained from the graining curve is presented in Table 1 and Figure 1.

![Figure 1. Grain-size distribution](image)

### Table 1. Properties of soil

<table>
<thead>
<tr>
<th>$c_v$</th>
<th>$c_u$</th>
<th>$D_{50}$</th>
<th>$D_{10}$</th>
<th>$D_{30}$</th>
<th>$D_{60}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.04</td>
<td>20.6</td>
<td>0.27</td>
<td>0.15</td>
<td>0.22</td>
<td>0.31</td>
</tr>
</tbody>
</table>

As it is obvious, the soil type is SP. In addition, the results of relative density test revealed that the maximum and minimum dry soil density was 1.75 and 1.49 grams per cubic centimetre, respectively. The density of the soil in the tank, which was poured from the raining sand machine, was measured to be 1.65 grams per cubic centimetre. Accordingly, relative density is obtained at about 65%. This relative density is suitable for our test because in very dense or very loose sand the change of distance does not have an impressive effect [9].

1813
\[ Y_{d(average)} = 1.65 \frac{gr}{cm^2} \]  

(1)

Direct shear test was performed to determine the internal friction angle of the soil which was \( \varphi = 38 \).

![Figure 2. Equipment of experiments](image)

The internal friction angle of the sand is directly related to the relative density. Bowles (1996) converted the relative density to \( \varphi' \) using the suggested following equation. The obtained friction angle and the relative density seem to be correct since they match Equation 2 [10].

\[ \varphi' = 28 + 15 \, D_r \]  

(2)

2.2. Testing Equipment

The testing set consists of two separate parts. One part is a tank which contains the soil of experiment is connected to the loading arm. The dimensions of the tank are 800 \( \times \) 800 and 600 mm in depth. The other part is deployed for sand raining located above the main tank. As shown in Figure 2, a hydraulic piston is responsible for providing the reciprocating movement of the second case. The reason for using a sand pluvial/raining method is to create homogeneous circumstances for tests. The relative density of the sand in the raining operation system is essentially dependent on the height of the pouring and the rate of sand discharge. The height of the sand raining in the tests is constant and the rate of the sand discharge is controlled by the opening of valves of the case. As mentioned, the density of the soil in the tank was 1.65 grams per cubic centimetre.

The wall of the tank could be influential in the stress and settlement of the pile group. Friction between the soil and container wall can reduce the vertical stress in the sand. This friction can result in the vertical stress being transferred to the wall [11]. In fact, the effect zone of the wall varies depending on the soil density and the instalment method of the piles [12]. According to the testing dimensions, the distance between the wall and the piles is 13 times bigger than the diameter of the piles, which is satisfying.

2.3. Model piles

The piles used in experiments are equal. These piles are steel and with the intention of reducing their weight, hollow ones have been used. The outer diameter of the piles is 22.2 mm, the inside diameter is 16 mm with the useful length of 30 cm. Based on the dimensions presented, the ratio of length to diameter of the piles is evaluated to be 13.5.

![Figure 3. Model piles](image)
The top part of the piles (4 cm length) is used to fix them to the cap and the grip set. Since the roughness of the piles significantly influences the shaft resistance and confining pressure of soil around the pile [13, 14], the outer part of the pile is covered with sandpapers as shown in Error! Reference source not found., in order to mobilize the shaft resistance and confining pressure of soil around the pile [13, 14]. Even though, the dimensions of test are not equal with real size in practice, the behaviour and mechanism of the reactions between piles and pile-soil are the same. The tests are designed in the biggest possible size and the soil and shaft material have been chosen from materials that can behave like a real size in practice.

It is demonstrated that in 4-pile groups, all of the piles endured the identical force and the maximum difference was between 3% and 7% in piles. There was a significant difference in the 9-pile group and the central pile tolerated 36% more force than the average bearing [3, 16]. Therefore, the arrangement of the piles in all cases was $2 \times 2$, so that the load is spread homogenously between the piles and the possibility of interruption by other factors was eliminated.

In these experiments similar caps have been designed for four modes. Caps thickness was 5 mm, and their size 20 to 20 cm. To have a uniform settlement, the cap should have rigid behaviour. In this regard, using the SAP2000 program, cap behaviour was modelled at the worst possible mode and the deformation rate was less than 0.01 mm. Therefore, it can be said that the cap behaviour was rigid.

The free standing mode is considered in these experiments since Lee and Chung (2005) proved that the interaction among piles is severely affected by the cap in comparison to a free standing group. Indeed, he showed the pile group capacity with cap remarkably increased at narrow pile spacing of 2d and 3d and then dramatically decreased at widening pile spacing [17].

2.4. Test procedure

The loading of the experiment has continued with consecutive imposing of weights which were 1 kg, till reaching 20 mm settlement. In regard that loading speed is effective in the bearing capacity, the loading speed during all experiments was constant and slow [18]. A load cell was set to record and send pressure of loading to the pc prepared for these experiments. In the same way, the settlement was recorded by a LVDT (Figure 2). All the data were saved in the pc in a special program to analyze.

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In order to investigate the effect of distance in the behaviour of the pile group, experiments were carried out in 4 different spacing including 3, 4, 5 and 6 times the diameter of the pile. It is worthwhile mentioning that the maximum value was considered 6 times pile diameter because from this point on, piles start to take the form of the single-pile. In addition, further spacing may cause interference of the tank-wall. It is noticeable that the distance should not be less than 2 times the diameter of the pile too since the soil loses its integrity and there would not be the desired interaction between soil and pile.
Each of these experiments was repeated 2 times. Sometimes, when it seems that the existing errors had a significant impact on the test results, the experiment was repeated and the test result which was more different from others is omitted. Single-pile experiments have been repeated 3 times. The intention of the experiment repetitions was reducing the error effect and reaching the most actual response. Although it is done to reduce the error to the lowest possible extent, the results are not error-free. The grip set, used to hold piles, causes inconsistency in pouring of sand, among the piles, which could reduce soil density of that area.

3. Results and Discussion

Single-pile tests continued to 1.5 cm settlement since single piles were ruptured in this settlement. As deduced from the figures, when the pile settles 1.5 centimetres, it comes close to the full- failure and loses its resistance against the load. The bearing capacity of the pile is about 20 kg and settlement is almost 3.2 mm. It should be noted that the bearing capacity is approximately evaluated through the fitting curve by Double Tangent method.

As it is obvious, the results of the tests of the pile group show less fluctuation than the single-pile. It can be stated that due to the high bearing capacity of the pile group compared to a single-pile, the effect of the errors is insignificant. The load-settlement curve of pile groups are shown in Figures 4 to 7.

The results show that in the spacing of 6 times the diameter, the efficiency coefficient reaches less than 1, but considering the error factors, it can be stated that the efficiency coefficient in the spacing of 6 times the diameter is also approximately 1. In fact the grip set in pile group is bigger causing some inconsistencies in soil which make the soil around the shaft looser than the single ones. Therefore the coefficient efficiency is smaller than 1. The efficiency of pile groups is more than single pile and this increase maximizes at closer distances. These results are similar to the results which previous researchers have achieved such as Vesić (1969) [3], Sales et al. (2017) [19], Lee and Chung (2005) [17]. In groups with distance of 3d, piles have interaction to each other. This interaction is a function of density and friction angle. In practice, the driven pile raises the soil density, and consequently increases the bearing capacity of the pile group. On the contrary, in situ pile is accompanied with well digging, resulting in looseness and decreasing the soil density. In addition, the materials used in this method like bentonite intensify the looseness. These two methods of installation have different effects on interaction factor [19-20]. In these experiments, none of these conditions was exactly dominated since piles initially were set. Then the soil was poured in the tank. In other words, the soil around piles is not sensibly disturbed.

The previous researches show that shaft resistance is not influenced by the installation method. Le Kouby et al. (2013) proved that the maximum resistance of pile tip in jacking installation is remarkably higher than non-displacement pile. However, in terms of shaft friction, despite the first phase of non-displacement loading test showing a stiffer response, the maximum values are similar [21].

As mentioned the tip resistance in pouring system is not so sensible. Consequently, it seems that shaft resistance is the most important item in groups in sand. In fact, the simultaneous presence of all piles in the soil mass increases the hardness of the enclosed soil, and as a result, more friction forces are mobilized. The small distances more intensively enclose the soil so it has no place to escape. The trapped soil under imposed loading settles and the density of it increases, which results in higher lateral earth pressure, so that the resistance of shaft would increase [22]. In other words, interaction between piles helps each other to endure more. With increasing of spacing, interaction between piles decreases and therefore the bearing capacity of the soil containing the pile group also declines. In spacing of 6 times pile's diameter, the efficiency coefficient decreases to the point that piles turn out to be single piles which perform separately. In that case the efficiency factor is almost one. It is also noticeable that the rate of the efficiency change is interesting. As seen the difference of efficiency coefficient between groups with distance to diameter ratio of 3 and 4 is 19% whereas the its difference, between groups with spacing-diameter 5 and 6 is 11%. This shows that interaction factor is more effective in closer distances and the change of distance impresses the bearing capacity more intense in small spacing. It seems that shear stress in shaft is mobilized more so the resistance increases intensively.

Moreover, it seems that with increase of the distance, the amount of settlement is declining. Actually the pile group has desire to show more settlement than single pile due to larger width that the pile group has. In lower spacing, the pressure bulbs produced by every pile overlap each other and cause a higher stress imposed to soil which result in higher settlement. In larger distances, the amount of settlement is restricted to constant amount.
Figure 5. Load-settlement of $s/d = 3$

Figure 6. Load-settlement of $s/d = 4$

Figure 7. Load-settlement of $s/d = 5$
Table 2. Efficiency coefficient at different spanning

<table>
<thead>
<tr>
<th>Pile group with ( s/d )</th>
<th>Force (kg)</th>
<th>Settlement (mm)</th>
<th>Efficiency coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>112</td>
<td>7.31</td>
<td>1.40</td>
</tr>
<tr>
<td>4</td>
<td>97</td>
<td>6.9</td>
<td>1.21</td>
</tr>
<tr>
<td>5</td>
<td>84</td>
<td>6.2</td>
<td>1.06</td>
</tr>
<tr>
<td>6</td>
<td>76</td>
<td>6.2</td>
<td>0.95</td>
</tr>
</tbody>
</table>

4. Conclusions

Based on the findings of the current study it can be concluded that:

- The spacing between the piles has a significant effect on the efficiency coefficient of the pile group.
- In general, the behaviour of the pile group embedded in sand is such that the efficiency coefficient would be greater than one.
- As the spacing of the piles increases, the efficiency coefficient of the pile group decreases. This value is equal to 1.4 for a spacing of 3 times the diameter of a pile. As the distance-diameter ratio increases, this value reaches 0.9 in the spacing of 6 times the diameter of the pile, which is close to 1.
- The change in distances in closer spacing is more effective on bearing capacity than in case of far spacing.
- With increase of spacing, the settlement decreases since interaction between piles decline which always is higher than single pile.

5. Acknowledgement

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6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Field Assessment of Non-nuclear Methods Used for Hot Mix Asphalt Density Measurement

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Abstract

Destructive nature along with the associated higher cost of the traditional core method used for hot mix asphalt density measurement has convinced researchers switching to some non-destructive technique for this purpose which is cost efficient as well. Earlier, nuclear density gauges were introduced for this purpose which was non-destructive as well. Since such devices were associated with the use of gamma rays, therefore, leading to safety and health issues. Last decade observed a revolution in asphalt density measurement technique with the evolution of non-nuclear density gauges. This research work is carried out with the objective to determine the efficiency and accuracy of a newly developed non-nuclear density gauge i.e. PQI-380 for field conditions as it needs its thorough evaluation prior to future uses in many of the developing countries including Pakistan. Density data obtained using standard core method and non-nuclear density gauge for 195 location confirms the satisfactory performance of the instrument. Results obtained show that the coefficient of correlation is near to 0.9. Which refers to a strong correlation between the density data. Moreover, performance criteria e.g. root mean square error and mean absolute error between the density data set is also very low confirming the good measuring abilities of the device. Instrument performed well for repeatability analysis giving maximum coefficient of variance less than 5 percent.

Keywords: Core Method; Non-nuclear Density Gauge; PQI-380; Density Measurement.

1. Introduction

The density of hot mix asphalt (HMA) has vital importance in paving industries because of many prominent reasons. One reason, for example, includes challenges involved in maintaining, rehabilitating and managing the pavement structure due to the aging of the roads, and budget consideration for the developing countries in particular and the developed countries in general [1].

Moreover, the density of hot mix asphalt is the decisive factor in predicting the future pavement failure as a low density may increase the rate of deterioration and may have the chance of oxidation to occur moisture issues and may lead to cracking and raveling [2-6]. On the other hand, density values that are higher enough thereby reducing the air voids content less than 3 percent may cause premature rutting [7]. Since the density of HMA plays a vital role in funds allocation for maintenance program along with its direct impact on the performance of the structure thus gaining a vital position among the researchers.
Many of the developed countries have started using few non-destructive and latest techniques for density determination but still Pakistan along with many of the developing countries are using the traditional destructive core method for this purpose. Core method which is proceeded in accordance with American association of state highway and transportation officials (AASHTO) procedure AAHTO T-166 is associated with extracting the cores thereby associated with disturbing the pavement integrity [8]. Secondly, the core method of density determination is associated with time limitation as we need at least 24 hrs. to make the core sample air dried before measuring the density and artificial. Drying in an oven at the elevated temperature may distort the core sample thereby actual results may change [7].

Because of many problems associated with core method, researchers have always been in search of some method which do not require disturbing the pavement integrity. Nuclear density gauges were the first most developed way of HMA density measurement. A major problem associated with such kind of gauges involves health risk as they work on the principle of sending and receiving the scattered gamma rays [9]. Because of health risk associated, working with such gauges requires a very strict licensing. Moreover, these gauges were quite heavy in the past and operator need a scale to measure the density from a safe distance associated with the risk of health issues [10].

Despite being free from the problem of disturbing the road integrity, nuclear density gauges are said to be not as accurate as that of core method. The research concluded that five different models of nuclear density gauges showed different results varying from place to place and even from model to model at the same place [11]. Nuclear density gauges have been used for many years in many countries across the globe but still, there was a need for an equipment which is not destructive thereby bypassing core method along with being free from health issues as in case of nuclear density gauges [12].

Paving industry has seen revolutionary changes in the last ten years in the field of density measurement as industry witnessed non-nuclear density gauges. Measurement principle of Non-Nuclear Density Gauge (NNDG) is based upon sending and receiving electromagnetic waves thereby beating the issues related to health as in case of nuclear devices and disturbance of pavement integrity as in the case of core method [13].

This research work is done to assess the abilities of Electromagnetic, non-nuclear density gauge used for HMA density determination. This research work is primarily influenced by the research conducted to check the performance of non-nuclear density gauges. One research work has concluded that the density measured using one of the non-nuclear density gauges i.e. PQI-380 is statistically very different in comparison to standard core method [13]. Other research done to compare the results of three different non-nuclear devices to that of nuclear density gauges suggested that nuclear density gauges, on average, read the density value on the higher side. Research also concluded that some of the factors i.e. air voids, specific gravity, and pavement layer thickness affected the density reading determined through both the nuclear as well as non-nuclear density gauges [14]. Other research concluded that few factors including orientation of the gauge, moisture presence and marking paint have no significant impact on the gauge reading [15]. One of the researches done on non-nuclear density gauge compares the cost-effectiveness of both nuclear and non-nuclear methods and reaches to a conclusion that non-nuclear density gauges are more cost-efficient in comparison to nuclear ones [16].

One research conducted on non-nuclear density gauge stated that the number of the core can be reduced to a much lower level by the usage of such gauges [17]. In a study of Rogge and Jackson (1999), it is concluded that none of the density gauges has proved itself to be accurate enough to replace the standard method of density determination i.e. core method [18]. Non-nuclear, electromagnetic density gauges are said to be standard equipment for asphalt density measurement only when they produce results comparable to AASHTO T-166 [19]. One research work is carried out to check the efficiency of a non-nuclear method to assess the compactness uniformity of recycled asphalt pavement. The results have concluded that HMA density can only be predicted accurately if HMA is prepared as per the guidelines of manufacturer of the instrument. Moreover, instrument reduced the number of cores drilled out as the compactness of the pavement was assessed to a good level of accuracy using step frequency radar [20]. Leng et al. (2018) evaluate the non-nuclear density gauge under various laboratory-controlled conditions have concluded that the direction of placing the instrument has no effect on density readings. They also concluded that gauge efficiency is affected by different gradations, but it has no clear effect by the bitumen content [21]. Van den Bergh et al. (2017) carried out to monitor the compaction process concludes that immediate evaluation using density gauge is important to get accurate results. Moreover, this method is cost efficient and requires a smaller number of cores for density measurement. This method has also the associated disadvantages of training a person for density measurement along with the possible error in reading due to many local factors including temperature and moisture variations [10].

Abyad (2016) evaluate different non-nuclear devices for density determination concluded that non-nuclear density gauges are moisture and temperature sensitive. Their efficiency is greatly influenced by the gradation along with variation in the local climatic conditions [22]. Another research carried out to compare nuclear and non-nuclear density gauges concluded that although no-nuclear density gauges are more affected by the moisture present on the pavement surface but at the same time the results obtained by non-nuclear density gauges are more reliable in comparison to nuclear density gauges [9].
The present work is primarily influenced and inspired by the fact that the pavement industry is on a verge to maturity across Pakistan thereby enhancing the need for some accurate, rapid, non-destructive and cost-efficient method for HMA density measurement. Main objectives of the research include the following:

- Validation and evaluation of the non-nuclear density gauge in field conditions
- Performance verification of non-nuclear density gauges in comparison to standard core method.
- Establishing statistical correlation model between the density data obtained from both the core method and non-nuclear gauge to predict the core density from the non-nuclear density data.

Taxila institute of transportation engineering (TITE), to accomplish the above stated objectives arranged one of the latest available non-nuclear density gauges i.e. PQI-380 which uses the concept of impedance spectroscopy to measure the asphalt density. The results obtained from PQI-380 were compared to already existing standard core method to validate the efficiency of the gauge.

2. Research Methodology

This research work is designed to achieve the stated objectives: to validate the equipment under field conditions; and, to compare the density data obtained from the gauge and core method. In the first step of research, density data were obtained from NNDG from the selected sites which consist of freshly laid asphalt pavement as well as aged structure. From the same location, where density was determined using NNDG, we extracted the cores and measured the density in the laboratory. Finally, both the density data were compared, and a statistical correlation was developed so that one can predict the core density even without extracting the cores in real practice. In the coming lines, a brief introduction to the working principle of non-nuclear density gauge is explained. Then the site selection for density determination, general characteristics of the material used in the pavement structure is described.

2.1. Non-Nuclear Density Gauges

PQI-380 determines the density of asphalt pavement by emitting the electromagnetic rays. These rays when hit the pavement surface i.e. HMA which is nonconductor material losses its strength. The amount by which this electric field reduces its strength is referred to as dielectric constant of the material. Density estimation by the gauge is carried out by the reduction in electromagnetic rays strength which is displayed on the screen [23, 24].

Non-nuclear density gauges on average have similar characteristics. Major characteristics of the non-nuclear density gauge used in this research work are enlisted in Table 1.

<table>
<thead>
<tr>
<th>Table 1. General characteristics of PQI-380</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General specification of NNDG</strong></td>
</tr>
<tr>
<td>Area of sensor</td>
</tr>
<tr>
<td>Depth range</td>
</tr>
<tr>
<td>Temperature sensitivity</td>
</tr>
<tr>
<td>Accuracy</td>
</tr>
<tr>
<td>Screen display</td>
</tr>
</tbody>
</table>

Operation principle of electromagnetic density gauges i.e. Pavement quality indicator (PQI) is shown in Figure 1 where a transmitter is shown from which the rays are introduced to the pavement surface and the other one being the receiver that collects the scattered rays. Pavement quality indicator works on a constant voltage, low frequency, electrical impedance approach. The density of HMA measured using NNDG is highly affected by the types of material used in asphalt mix as HMA comprise of many components and for each component, dielectric constant varies considerably e.g. air, 1; water, 80; aggregate, 4-20) [25].
Non-nuclear density gauge i.e. PQI-380 used for this research work is shown in Figure 2.

2.2. Site Selection for Field Studies

This section elaborates the sites selected for the field testing. First sites selected for performance verification is explained and then basic and fundamentals of the selected sites i.e. aggregate gradation along with properties of the binder materials are also explained. Field studies were done to compare the densities obtained using NNDG and Core methods. It included a total of 195 cores extracted from two different sites. One of the sites was near Sahiyan wala interchange on Faisalabad Multan motorway M-4. Total of 150 cores were extracted from this location during high-temperature conditions. Aggregate gradation used for this site was dense gradation. Google map image is inserted for the first site as shown in Figure 3 While another site was a service road in Rawalpindi Gulberg green. Total of forty-five cores were extracted from this site during low-temperature conditions. This road was also constructed using dense gradation and cores were extracted for wearing course. Google map image for this site is inserted as shown in Figure 4.
As it is already discussed that for both the selected sites, dense graded aggregates are used as per NHA-B. Gradation curves for NHA-B is shown in Figure 5.
A flow chart is shown in Figure 6 that describes the methodology of this research work. The research includes the testing that covers checking the effect of calibration for field density determination. Purpose of doing calibration is to increase the accuracy of the results obtained from field conditions. Effect of calibration is checked by preparing the slabs in the laboratory so that possible effect can be accomplished.

![Flow chart describing the methodology of research work](image)

### 3. Results and Discussion

Major outcomes from the tests carried out in order to meet the stated objectives are described in this section. First, the density data from both the technique is compared and checked against whether the results are statistically similar or not. Moreover, for both the sites a statistical correlation model is developed so that one can predict the core densities without extracting the cores for future work. In this way, this study clearly gives a relation based on correlation developed for a huge number of extracted cores.

#### 3.1. Field Results

For field assessment of the non-nuclear density gauge total of 195 locations were selected and the density data using the gauge is determined. Secondly at the same location cores were extracted and the density is also measured in the laboratory as per AASHTO T-166. Density determination using NNDG and core method is shown in Figure 7 (a) and (b) respectively.

![Density determination using two methods: NNDG (a) and drilled cores for laboratory density measurement](image)
A linear regression model is developed from the field data in order to check whether the results from both the methods are comparable or not. Moreover, on average, how the NNDG reading is varying for both the hot and low-temperature range is explained and elaborated and lastly, a correlation equation is developed for both the temperature conditions so that one can predict the core density even without extracting the cores. The basic purpose behind the development of the equation is to minimize the core extraction.

For site 1 i.e. Sahiyan wala interchange on M-4, It is shown in Figure 8 where a scattered chart is plotted between the two densities data. The graph has density measured using NNDG on Y-axis while X-axis carries the density data obtained from the core method. Core density being the independent variable is plotted on X-axis while NNDG being a dependent variable is plotted on Y-axis. This plot confirms that under high-temperature conditions NNDG performance is satisfactory as the value of the correlation coefficient is 0.752 which means that using this model 75.2 percent of core density data can be predicted accurately and vice versa. For the best correlation, this coefficient is said to have the value near or equal to 90 percent. At the same time, the value of 0.752 is acceptable as the data is for 150 points. Hence, it can be concluded that 75.2 percent of accurate prediction of density data is enough to say that NNDG i.e. PQI-380 has performed well for field conditions of high-temperature range.

Similarly, a correlation model is developed for the second site i.e. service road at Gulberg green as shown in Figure 9. For this site, a correlation coefficient is on a higher side which is 0.82. The value of correlation coefficient confirms the satisfactory performance of NNDG under low temperature as well. Therefore, from these results, it can be concluded that NNDG performed satisfactorily under field conditions of varying temperature.

Figure 8. A linear model developed for Site 1; Sahiyan wala interchange (Faisalabad Multan motorway)

Figure 9. A Linear Model developed for site 2; Service road west Gulberg green
The results obtained from developing a correlation says that on average for both the sites, PQI-380 has shown results that are quite correlate able to already existing standard core method. therefore, such gauges have a bright future to be used in the field of asphalt density measurement as a sole replacement to core method. The field results were analysed in a different perspective to check whether the mean of the two methods are statistically different from each other. For this purpose, bar charts are plotted for both the sites which contain average density on Y-axis while two bars are shown representing both the methods. It is shown in Figure 10 that NNDG value is on lower side in comparison to that of core method. The percentage difference between the density data is only 1.79 percent. While on the other hand for site 2 i.e. Service road near Gulberg green it can be seen form the Figure 11 that NNDG reads density value higher in comparison to density data obtained from core method. The percentage difference between the density data is 4.01 percentage.

![Figure 10. Average density data for site 1; Sahiyan wala interchange (Faisalabad Multan motorway)](image1.png)

It has been concluded by the researchers that the temperature may affect the electrical conductivity of asphalt concrete [25]. Average NNDG density to be on the lower side for site 1 where the temperature was on a higher side and to a higher side in the case of the second site, where the temperature was on the lower side, can be explained by the above-mentioned temperature effect on the electrical conductivity of asphalt. Moreover, researchers have also stated that different material may exhibit different rate of change in dielectric constant for same temperature variation [26].

![Figure 11. Average density data for site 2; Service road (gulberg green)](image2.png)

Accuracy of non-nuclear density gauge can be improved if the temperature sensitivity of the instrument is decreased. Although it has shown the results that are comparable with the results obtained using the core method still there is a need to determine the effect of temperature on the gauge reading. The efficiency of the instrument can be enhanced by making the instrument less temperature sensitive.

It can be seen from the Table 2 maximum mean absolute error (MAE) for both the sites is 0.015 while other performance evaluation criteria also states the satisfactory performance of the instrument.
Table 2. Performance criteria for both the sites

<table>
<thead>
<tr>
<th>Performance criteria</th>
<th>Site 1 Model data set</th>
<th>Site 2 Model dataset</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r^2$</td>
<td>0.752</td>
<td>0.82</td>
</tr>
<tr>
<td>RMSE</td>
<td>0.009</td>
<td>0.012</td>
</tr>
<tr>
<td>MAE</td>
<td>0.008</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Root mean square error in the data set is even below 5 percent that means the PQI-380 has a huge potential to be used for asphalt density measurement.

3.2. Predicting the Core Densities

One of the objectives of this research includes minimizing the number of cores extraction for future work so that pavement integrity is disturbed to a minimal level. For this purpose, this research suggested two equation which holds good for two temperature ranges i.e. lower and higher one.

Equation 1 holds valid for lower temperature conditions that has temperature range of 0 to 15˚C while Equation 2 is valid to predict core density from NNDG data in high temperature conditions ranging from 15 to 35˚C.

$D_{Core} = 0.9396 \times D_{NNDG} + 0.2308$  \hspace{1cm} (1)

$D_{CORE} = 1.2918 \times D_{NNDG} − 0.4767$  \hspace{1cm} (2)

Where $D_{Core}$ = density of the asphalt pavement measured using core method (g/cm$^3$) and $D_{NNDG}$ = Density of the asphalt pavement measured using NNDG i.e. PQI-380 (g/cm$^3$)

This research work has a prime objective of decreasing the number of core extraction as this activity has long-lasting effects on the pavement structure and it disturbs the pavement integrity. These two equations were developed so that core density for any of the pavement can be predicted just by knowing the density of that point using NNDG under the varying condition of temperatures.

3.3. Repeatability Analysis

Repeatability of the EM density gauge was verified by measuring the density of each location for at least ten times. Each time instrument was lifted 2 inches high and then placed again at the same point. For better graphical representation five locations were set for both the sites and density was measured ten timed. Repeatability test for site 1 and site 2 is shown in Figure 12 and 13 respectively.

![Figure 12. Repeatability analysis for site 1; Sahiyan wala interchange (Faisalabad Multan motorway)](image-url)
Results indicated that for site 1 at Sahiyan wala interchange prepared as per NHA-B having high-temperature conditions, the coefficient of variance comes out to be 0.12 percent as shown in Figure 14. This value is much lesser indicating the instrument has results comparable enough as it will show almost similar results taken at a different time for the same location. Similarly, for the second site at service road west near Gulberg green, the coefficient of variance was 0.21 percent as shown in Figure 14. This value showed that for the repeatability, instrument performed well enough. In comparison to the first site (high-temperature conditions) results show that performed of non-nuclear density gauge is way better than for low-temperature condition as for repeatability coefficient of variance is less.

4. Conclusions

This research work covered performance verification of non-nuclear density gauge for field condition. Major results obtained from the research are elaborated below:

- Field study conducted to evaluate the performance of non-nuclear density gauge concludes that the instrument performed well under field conditions of varying temperature. Coefficient of correlation from both the sites are near to 0.9 on average making it more comfortable to rely on the performance of NNDG.

- Maximum percentage difference between the density data obtained from both the sites confirms that this difference is even less than 5 percent.
• This research work highly recommends use of NNDG i.e. PQI-380 for asphalt density measurement for future only if the results are comparable to that of already established standard core method results.

• Repeatability analysis have shown that NNDG produces similar results measured for same location at different times. This thing encourages the author to conclude that PQI-380 produces more consistent results.

This study has recommendations to go for a detailed evaluation of such gauges involving their cost effectiveness. Such gauges should be evaluated based on their performance under varying climatic conditions e.g. temperature, moisture and different raw debris presence on the pavement.

5. Acknowledgement

All praise to Almighty Allah, the creator of everything. Authors show huge regards for national highway authority (NHA) of Pakistan for allowing the access to the core extraction. Authors also acknowledge the efforts of capital development authority (CDA) for providing the density data for cores from service road west near Gulberg green, Islamabad, Pakistan.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Optimum Efficiency of PV Panel Using Genetic Algorithms to Touch Proximate Zero Energy House (NZEH)

Bdoor Majed Ahmed, Nibal Fadel Farman Alhialy

Abstract

By optimizing the efficiency of a modular simulation model of the PV module structure by genetic algorithm, under several weather conditions, as a portion of recognizing the ideal plan of a Near Zero Energy Household (NZEH), an ideal life cycle cost can be performed. The optimum design from combinations of NZEH-variable designs, are construction positioning, window-to-wall proportion, and glazing categories, which will help maximize the energy created by photovoltaic panels. Comprehensive simulation technique and modeling are utilized in the solar module I-V and for P-V output power. Both of them are constructed on the famous five-parameter model. In addition, the efficiency of the PV panel is established by the genetic algorithm under the standard test conditions (STC) and a comparison between the theoretical and experimental results is done to achieve maximum performance ranging from 0.15 to 0.16, particularly with an error of about -0.333 for an experimental power of 30 Watts compared with the theoretical power of 30.1 Watts. The results obtained by the genetic algorithm give the best value for efficiency at the range of 16% to 17% of solar radiation, from 500–600 W/m². These values are almost identical to the efficiency obtained from the results of the operation, where the best value for efficiency in the experimental results was seen to be 15.7%.

Keywords: Genetic Algorithm; Optimum Efficiency; Photovoltaic Panels; Model of Single-Diode.

1. Introduction

Due to the increasing demand for clean and renewable energy applications, to avoid increasing CO₂ release into the environment, there is a growing demand for electrical energy, specifically in Iraq. Solar energy is plentiful in Iraq throughout the year. In Iraq, there is a leak in the provision of electricity, especially in the summer season, coinciding with the high rising temperature and increasing demand for cooling. Iraq is qualified to be a producer of all solar applications and is expected to provide solar electricity to its neighbours. Academicians in Iraq have investigated the effects of weather conditions on different solar energy applications in detail and obtained significant results, possibly the crucial point is that the weather conditions in Iraq fit all solar applications [1]. Near Zero Energy House (NZEH) or Net Zero Energy Building (NZEB) design is a housing building that provides energy efficiently by using renewable energy technologies and passive house design. The basic conception of Zero Energy Building (ZEB) is that a building should produce its energy from clean, renewable, and less expensive sources. Presently, the budget for NZEHs is fairly high owing to the high costs of the apparatus and resources for solar sheet, isolation, fenestration, and other renewable energy expertise. As a result, an investigation to attain the optimum design of an NZEH is obligatory. The aim of the optimal design is to accomplish a reasonably priced human life progression budget performance of the NZEH. The Genetic Algorithm is one of the best optimization techniques that can be utilized. It provides a technique to achieve the...
optimal strategy established on the variable design groupings of NZEH. The investigated design parameters include photovoltaic (PV) array efficiency optimization [2, 3]. For designing a solar photovoltaic system for a single family dwelling, the civil engineer must design the structural engineering or architecture for PV supplemental information to construct a structure and a roof plan. He has to provide a roof plan projected on a site plan; show the location and dimensions of all solar photovoltaic equipment and PV arrays (Figure 1).

A Photovoltaic generator (PV) is a nonlinear system, because its behaviour depends mainly on environmental circumstances, for instance, radiation, temperature, wind speed, humidity, dust, clouds, etc. Alima (2015) and Zahiidee et al. (2016) have investigated the dependence of PV on the effect of temperature on the output power of different PV modules [4, 5]. For increasing the quality of the manufactured solar panels, devices, and modelling of the device and its simulators, careful extraction and an improvement of solar panel parameters is required. Direct methods depend on the use of the I-V curve, at specific points, to determine some cell parameters. Consequently the accuracy of these techniques is determined by the accuracy of the measured data and the data processed by the manufacturer, as also the errors made by the numerical and simplified discrimination of formulae used to extract the parameters. Other methods used for extracting solar panel parameters rely on the use of optimization algorithms to determine the solar cell parameters. Its precision depends on the application's installation algorithm, the user-defined error function, and the primary values of the parameters to be installed.

Ismail et al. (2013) and Jervase et al. (2001) used two models (distinct diode prototypical and dual diode prototypical) and three types of PV elements (monocrystalline, polycrystalline, and thin-film) and studied the effect of partial shading on the PV module, to simulate the parameter using Matlab-Simulink [6, 7]. The results obtained from Matlab-Simulink were compared with the results processed by the manufacturer’s data sheet and were acceptable. They employed the genetic algorithm to extract the optimal parameters from the photovoltaic module. Ramoji et al. (2014) and Ramoji and Kumar (2014) used the GA algorithm optimization technique to optimize the size and applied it to reduce the total cost [8, 9]. The suggested hybrid energy system consisted of PV panels, a wind turbine, and a storage battery. The result demonstrated that the GA was converging very well. Rodriguez (2017) used a single-diode with five parameters, such as, Iph, I sat, η, Rs, and Rh. These parameters changed with solar radiation and temperature; hence, the genetic algorithm was used to categorize these parameters and a monocrystalline PV panel was used. The major aim of this technique was to acquire a number of factors for dissimilar ecological circumstances, deprived of appliance, and slightly oversimplified. In this article has been emphasized that wholly the considerations alteration. To be exact, it has been displayed that η, Rs, and Rh presented important dissimilarities in the climatic surroundings. The chart of Rh displayed that its level is powerfully nonlinear with relevance given to irradiance and temperature variations; and also to dirtying, aging, degradation, etc. The values of the parameters within the PV model will settle for a sudden drift, so the recommended methodology may be used as an orientation for pinpointing a purpose, because of its ability to use only five operating points for accurately identifying all parameters of the single-diode model. Zagrouba et al. (2010) studies, depend on the approach of the genetic algorithm to identify the parameters of the PV cell and PV module (Is, Iph, Rs, Rash, and n) [11]. These parameters are used to find maximum power. They used the Pasan’s software to obtain the parameters of cells and used the GA to obtain the cell electrical parameters from the determined I-V curve. Then it compared these results with those obtained by using the Pasan’s software. Simulation results offered that the accuracy of the fitting approach, in the case of solar cells, was better than that of the solar modules, where the values of the minima were shown to be equal to 0.000256 A2 for the solar cell and 0.0676 A2 for the module. Eimhjellen (2018) developed an algorithm for calculating the optimal solution for the design of a solar farm with fixed panels [12]. Ayman et al. (2017), Rahmani et al. (2018) and Pereira and Aelenei (2019) presented an evaluation of the state-of-the-art scientific accomplishments for using Artificial Intelligence (AI) techniques in Photovoltaic (PV) structures. It analysed the role of AI algorithms in modelling, sizing, control, fault conclusion, and output assessment of PV systems [13-15].

In this study, the behavior of solar cells and modules under numerous operating conditions will be determined effectively, once their intrinsic parameters are measured accurately and are calculable. The aim is to simulate the current–voltage (I-V) characteristics, a point supported on genetic algorithms by Matlab. The plan is to extract the optimum potency of the photovoltaic panel and to enhance the accuracy of the solar cell parameters, which can be gained by exploitation of direct techniques and an experimental study. The idea is that the formulate of photovoltaic cell effectiveness, extraction, and optimization problems of the single diode model, wherever the greatest equation of the current is utilized in genetic algorithms, and also to introduce minimum and maximum assessment, such as, a shunt and a series of resistance extracted by mathematical equations. Subsequently the examination will give the efficiency gained from the genetic algorithmic program, along with the efficiency gained from experimental study.
Figure 1. PV location on the roof [16]

2. Mathematical Modeling of PV Module

The single-diode model has been used to investigate the relation between voltage $V$ and the current $I$ in Equation 1, it is the main equation that used in genetic algorithm, as given by El Tayyan (2015) [17], the modeling of the single diode corresponding circuit of a PV device is shown in Figure 2.

$$I = I_{ph} - I_o \left( \exp \left( \frac{V + R_s I}{a V_T} \right) - 1 \right) - \frac{V + R_s I}{R_{sh}}$$  \hspace{1cm} (1)

The PV model can calculate the output power by the simplest form Hashim and Talib (2018) using Equation 2 [18]:

$$P = IV = \left[ I_{ph} - I_o \left( \exp \left( \frac{V + R_s I}{a V_T} \right) - 1 \right) - \frac{V + R_s I}{R_{sh}} \right] V$$  \hspace{1cm} (2)

The diode quality factor $a$. It is equal 1.2 for monocrystalline solar cell [19]. The efficiency of a PV panel is defining the relation of the output power to the input solar power:

$$\eta = \frac{P}{G \times A}$$  \hspace{1cm} (3)

Figure 2. The single-diode model comparable circuit of a PV cell [20-22]
3. The Genetic Algorithm

Genetic algorithms (GAs) are natural galvanized algorithms that support the thought of Darwinian evolution, which embraced the Associate Degree Format Method. The basic components common to all GAs are: a fitness function for optimization, representational choice, crossover, and mutation operators [23]. To deal with the I-V curves digitally, an approach was prepared and the procedure was based on the genetic algorithm (GAs); Equations 1 to 3 were used, minimum and maximum values of series resistance were entered, shunt resistance, voltage, solar radiation, and photo current were generated, to obtain the best value for PV panel efficiency that competed with the experimental one. The Genetic Assembly began by generating an initial set of random probable resolutions (chromosomes) of the problems that had to be improved [24].

Each chromosome was consequently weighed through an adaptive function, to describe the validity of every potential explanation [25]. Next, the genetic agents were applied toward generating novel children; the choice to determine which fathers would be introduced to the new generation according to their adaptive values. Crossover was later used for new atomic products by regrouping data from the parents selected in the preceding stage. The mutation was applied to a variety of populations by changing the genetic structure of some people according to the rate of mutation to obtain a new individual with an ideal specification [23]. One can review the genetic program flow chart in Figure 3, which illustrates the basics of its operation. The optimization of multiple-variables of the selected design parameters in a single-family building, in temperate climate conditions was implanted by Ferdyn-Grygierek, and Grygierek (2017) [26] which analyzed the inspiration of four categories of windows, their dimension, construction, alignment, isolation of outside walls, rooftops, ground floor, and infiltration on the life cycle costs (LCC). Finally the optimal decision of the design parameters was implemented by using genetic algorithms by coupling the building performance simulation program EnergyPlus with optimization of the environment.

Li et al. (2017) reached to a fact that Genetic Algorithm (GA) is one of the universally used optimization algorithms for building applications and presented future guidelines for building investigation communal on in what way to put on GA for the optimization of building energy [27].

![Figure 3. Flowchart of GA. Chief Process is in dashed box, others are the election approaches for each process [27]](image)

4. Experimental Work

The experimental setup of the system is explained in Figure 4, the Monocrystalline-Si solar PV module, which has an area of 0.282 m². Detailed specifications of the PV panel are explained in Table 1; the solar module analyzer measures the current, voltage, fill factor, and efficiency of the PV module. The temperature sensor has been utilized to measure the module temperature, solar power meter is utilized to measure the global radiation incident on the PV module and put it in the same tilt angle of the monocrystalline PV 32°, toward the south.
Table 1. Module specifications at STC as existing by the industrialist

<table>
<thead>
<tr>
<th>Specification of the manufacturer</th>
<th>Value in units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rated power</td>
<td>30 W</td>
</tr>
<tr>
<td>Voltage At Maximum Power (Vmax)</td>
<td>17 V</td>
</tr>
<tr>
<td>Current At Maximum Power (Imax)</td>
<td>1.76 A</td>
</tr>
<tr>
<td>Open Circuit Voltage (Voc)</td>
<td>22 V</td>
</tr>
<tr>
<td>Short Circuit Current (Isc)</td>
<td>1.9 A</td>
</tr>
<tr>
<td>Normal Operation Cell Temperature</td>
<td>25°C</td>
</tr>
<tr>
<td>Module weight</td>
<td>3.5 Kg</td>
</tr>
<tr>
<td>Area</td>
<td>0.282 m²</td>
</tr>
</tbody>
</table>

5. Results and Discussion

5.1. Experimental Results

The Experimental Results show the efficiency of the PV module on solar radiation (G = 500 W/m²) for four months in Table 2. It shows the efficiency of the PV module between 10.8% and 15.7%. Figure 5 shows the experimental result taken from the solar module analyzer.

Table 2. The experimental result of radiation G =500 W/m²

<table>
<thead>
<tr>
<th>Month</th>
<th>Tc/°C</th>
<th>Ta / °C</th>
<th>Pmax /W</th>
<th>Imax /A</th>
<th>Vmax/V</th>
<th>Voc/V</th>
<th>Isc</th>
<th>η %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oct. 2017</td>
<td>33.8</td>
<td>27</td>
<td>15.32</td>
<td>0.91</td>
<td>16.85</td>
<td>20.39</td>
<td>1.018</td>
<td>10.8</td>
</tr>
<tr>
<td>Nov. 2017</td>
<td>28</td>
<td>20</td>
<td>22.13</td>
<td>1.277</td>
<td>17.31</td>
<td>20.66</td>
<td>1.379</td>
<td>15.7</td>
</tr>
<tr>
<td>Dec. 2017</td>
<td>25</td>
<td>17</td>
<td>17.08</td>
<td>0.917</td>
<td>18.1</td>
<td>21.49</td>
<td>1.004</td>
<td>12.1</td>
</tr>
<tr>
<td>Jan. 2018</td>
<td>21</td>
<td>12</td>
<td>16.6</td>
<td>0.936</td>
<td>18.25</td>
<td>21.64</td>
<td>1.025</td>
<td>11.7</td>
</tr>
</tbody>
</table>

Figure 5. The P-V and I-V curve taken from the solar module analyzer
5.2. Genetic Result

In this section, the efficiency of the solar panel was calculated in the genetic program in the Matlab. Minimum and maximum values of series resistance, shunt resistance, photocurrent generated, the voltage of the PV module, radiation, and cell temperature were entered into the genetic algorithm at boundary conditions. Table 3 shows how the genetic program calculated the values. The optimum efficiency of the solar panel was obtained and was found to be in the range of 0.16 to 0.17, when the solar radiation was in the range of 500 to 600 W/m$^2$, such as is displayed in Figures 6 to 8. The optimum value of efficiency from this table = 17.03% and when there was an increase in the solar radiation the efficiency increased in this section. This can be observed in Figure 8. According to Figures 6 to 8, and for solar radiation from 600–750 W/m$^2$, shown in Figures 9 and 10, it was easy to observe that the optimum values of the efficiency obtained by the GA algorithm were, 16.9, 17.03, and 38.8%, respectively.

<table>
<thead>
<tr>
<th>Rs (Ω)</th>
<th>Iph (A)</th>
<th>Rsh (Ω)</th>
<th>Vpv (V)</th>
<th>G (W/m$^2$)</th>
<th>Tc (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max.</td>
<td>0.3</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>500.</td>
</tr>
<tr>
<td>Min.</td>
<td>1.9</td>
<td>1.9</td>
<td>500</td>
<td>22</td>
<td>600</td>
</tr>
<tr>
<td>1</td>
<td>1.7225</td>
<td>0.7824</td>
<td>100.0000</td>
<td>12.5365</td>
<td>599.9123</td>
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<tr>
<td>2</td>
<td>1.6823</td>
<td>0.8500</td>
<td>100.1145</td>
<td>12.0418</td>
<td>599.9610</td>
</tr>
<tr>
<td>3</td>
<td>1.6470</td>
<td>0.9626</td>
<td>100.0000</td>
<td>12.3183</td>
<td>600.0000</td>
</tr>
<tr>
<td>4</td>
<td>1.7118</td>
<td>0.7806</td>
<td>100.0518</td>
<td>12.5759</td>
<td>599.7605</td>
</tr>
<tr>
<td>5</td>
<td>1.6832</td>
<td>0.8719</td>
<td>101.0099</td>
<td>12.2369</td>
<td>599.9220</td>
</tr>
<tr>
<td>6</td>
<td>1.6838</td>
<td>0.7830</td>
<td>100.2250</td>
<td>11.1239</td>
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<tr>
<td>7</td>
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<tr>
<td>9</td>
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<td>0.7124</td>
<td>100.5751</td>
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<td>600.0000</td>
</tr>
<tr>
<td>10</td>
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<td>0.7202</td>
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<tr>
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<td>599.8159</td>
</tr>
<tr>
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<td>0.7619</td>
<td>100.0000</td>
<td>11.4137</td>
<td>599.6418</td>
</tr>
<tr>
<td>14</td>
<td>1.6852</td>
<td>1.0713</td>
<td>102.3651</td>
<td>12.2345</td>
<td>600.0000</td>
</tr>
<tr>
<td>15</td>
<td>1.6118</td>
<td>0.9476</td>
<td>100.0000</td>
<td>12.1988</td>
<td>600.0000</td>
</tr>
<tr>
<td>16</td>
<td>1.5803</td>
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<tr>
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<td>0.7218</td>
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<td>18</td>
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<td>0.7296</td>
<td>100.2915</td>
<td>12.3092</td>
<td>599.7827</td>
</tr>
</tbody>
</table>

Figure 6. The optimum value of efficiency of radiation (500-600 W/m$^2$)
Figure 7. The optimum value of efficiency of radiation (500-600 W/m²)

Figure 8: The optimum value of efficiency of radiation (500-600 W/m²)

Figure 9. The optimum value of efficiency of radiation (600-750 W/m²)
6. Conclusion

In this study, the modeling process was based on the genetic algorithm that had been proposed to obtain the optimal values for solar panel efficiency; a single-diode model was used. The efficiency values extracted by using this algorithm were viable for the entire range of solar radiation, temperatures, voltage, series resistance, and shunt resistance. The information on the PV module, presented in the manufacturer’s data sheet, was the requirement for this approach. The result validation was conducted for the type of PV module technologies, monocrystalline, by comparing it with the result of the experimental study, where the results of the efficiency of the panel, extracted from the genetic algorithm, ranged between 0.16–0.17, and the results obtained from the experimental study ranged between 0.108–0.157. This gave an indication of the accuracy and validity of the genetic algorithm.

This application is not confined to monocrystalline solar panels alone; it can be applied to all solar panels and can be utilized to extract all solar panel parameters. The slight differences between the experimental results and genetic optimization are due to the temperature variation effects, significantly on the output voltage, as the open circuit voltage has a logarithmic relationship with the inverse of the reverse saturation current, which is greatly influenced by temperature.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Effect of Viscosity Parameter on Numerical Simulation of Fire Damaged Concrete Columns

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Received 08 May 2019; Accepted 28 July 2019

Abstract

The assessment of the residual strength of post-heated concrete structural members in a professional way is a prime factor to take a decision about the restoration or destruction of fire-damaged structure. This Paper explores the numerical modelling of RC square columns damaged by exposure to heat at 500°C, unjacketed. Software ABAQUS was used for numerical modelling of fire damaged compression member i-e column. The main objective of this study is prediction of axial load and axial deformation of fire damaged concrete using finite element studies. Moreover, a parametric nonlinear finite element (FE) research is carried out to check the effect of viscosity parameters on numerical simulation of fire damaged concrete columns. For the said objectives, numerical simulation of existing experimental study of fire damaged RC columns is conducted with varied values of viscosity parameters. The numerical analysis (Finite Element Modeling) indicated that axial load capacity decreases and axial deformation increases after exposure to fire. The experimental and numerical studies are compared in terms of load displacement analysis. The use of optimum viscosity parameter and its definition to FEM improves significantly the performance of convergence and reduces analysis time of numerical simulations of RC square columns. Moreover, a good agreement was found between the experimental and the finite model results.

Keywords: Fire Damaged; Viscosity Parameter; Concrete Damage Plasticity; ABAQUS; Numerical Modelling.

1. Introduction

The international association of fire and rescue services report published in 2018 showed that more than 3 million fire incidents occurred around the world resulting about 18000 civilian deaths, 58.6 thousands civilian injuries and million dollars directly property damage. With such high figure, it is inevitable to develop a procedure to assess the residual performance of structural system after fire [1]. Reduction in performance of building material is noted after exposure to high temperatures [2]. The exposure of building to fire resulted in 58% reduction in strength [3]. The use of Finite element model (FEM) for prediction of axial capacities and deformations is also employed these days. Mohamed Bikhiet et al. (2014) used it to check the behavior of fire damaged columns. He concluded that due to increase in surface temperature faster failure occurred. Moreover, Ma et al. (2012) used it for numerical simulation of already performed experimental work. The concrete damage plasticity model was adopted for the calculation of constitutive concrete material used in the columns and the viscosity coefficient was discussed. It was specified that until finding a reasonable value for viscosity parameter, a parametric study should be conducted in order to improve convergence of numerical simulation. Two different constitutive material models are offered in ABAQUS/Standard, which is an implicit analysis program, for the analysis of concrete at low confining pressures: the smeared crack concrete model and the concrete damaged plasticity (CDP) model. Moreover, the CDP model is based on the degradation of the elastic stiffness induced by plastic straining both in compression and tension (assumption of isotropic damage) [4]. A non-linear FEM has been

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adapted by Rami H. for shear deficient heat-damaged concrete. He concluded that, for CDP material model it can be defined flow potential, yield surface, and viscosity parameters [5]. The numerical model developed for retrofitting of damaged structural member showed that the developed model can be an effective tool to predict the performance of retrofit beams under dynamic loading condition [6].

Piscesa et al. (2017) got excellent results in his study where the steel rebar is modelled as a truss element with elastic-perfectly plastic material behaviour. The concrete is modelled as 8-noded hexahedral element [7]. Accuracy and reliability are verified by simulating experiment on a plain concrete specimen. Two laboratory experiments consisting in pushing until failure two 2-D RC frames are simulated with the proposed approach to investigate its ability to reproduce actual monotonic behaviour of RC structures [8]. FE numerical simulation has been employed by many researchers to predict the behaviour of heat damaged RC columns with different wrapping materials and bonding dimensions [9-11]. Mohamed Bikhiet et al. (2014) checked the nonlinear behaviour of damaged concrete. He found that along with the application of load and increase in surface temperature the column failed faster. The simulation also showed the effect of temperature on stress and its distribution [12, 13]. ABAQUS is actually implicit analysis program which constitutive model. Two main models used in ABAQUS for modelling concrete are “Brittle Cracking Model for Concrete” and “Concrete damage Plasticity model” [14]. The concrete damaged plasticity model is most widely used model based on the assumptions of isotropic damage and degradation of elastic stiffness induced by plastic strain [15-20].

It has been observed from the literature that the finite element behavior of the Post heated unconfined concrete columns using software (ABAQUS) is less explored. There is a gap in literature about effect of viscosity parameter on numerical simulation of fire damaged RC Columns. An effort is made in this research to give a model for prediction of axial load and axial deformation of fire damaged RC Square Column and effect of viscosity parameter on numerical simulation. Therefore, to accurately analyze and stimulate such columns critical parameters that influence the axial capacity of RC columns needs to be studied.

1.1. General Analysis

Abaqus (FEM code) which is general purpose code was used for nonlinear analysis. The library of Abaqus contains several constitutive models and has a complete geometric modeling capability. Analysis follows several steps, each of which shows response simulation. This system also includes preprocessing and post processing techniques. FEM code can cope with coupled analysis, meaning temperature and displacements are integrated simultaneously.

2. Material Properties

2.1. Damaged Plasticity Model of Concrete

The plastic behavior of concrete can be defined by any of the following constitutive three models in ABAQUS, the concrete Smeared Cracking model (CSCM), Brittle Cracking Concrete (BCC) and Concrete Damaged Plasticity model (CDPM). The damaged plasticity model compacts with compressive, plastic, and tensile behavior and damaged mechanism of concrete. ABAQUS by default defines compressive, tensile and plastic behavior of concrete.

2.2. Viscosity Parameter (μ)

This is parameter that is used to prevent numerical instabilities and strain localization. The behavior of structural members that is column and beam actually define the behavior of whole structure. That is why nonlinear behavior of these members is very important for safe design of structures. Plastic, compressive and tensile behavior of concrete are the main inputs required in Plasticity model. This model (CPD) can be regularized using viscoplasticity by allowing stresses to be outside yield surface. Duvaut-Lions generalization is used, which states Visco-plastic strain rate tensor, $\varepsilon_{\nu}^{pl}$ as:

$$\varepsilon_{\nu}^{pl} = \left(\varepsilon^{pl} - \varepsilon_{\nu}^{pl}\right) / \mu$$

Here $\mu$ is viscosity parameter and $\varepsilon^{pl}$ is plastic strain. Small values of viscosity parameters are used to achieve good results. The general guideline given by Lee et al. (1998) is that it is taken as 15% of time increment [4, 21].

2.3. Damage Parameters

The ratio of cracking strain to the total strain is known as tensile damage parameter ($dt$). Similarly the ratio of inelastic strain to the total strain is compressive damage parameter ($dc$). If these parameters are not specified, the model is termed as plasticity model.
2.4. Dilation Angle

This is an angle of cracking of concrete. The value of dilatancy parameter $\alpha_p$ ranges from 0.2 to 0.3 [22, 23]. For specified range the dilation angle should be between 310 to 420. In this study dilation angle ranging from 30° to 45° were examined.

3. Experimental Study

An Experimental study conducted by Yaqub et al. (2010) is selected as reference study in order to create a numerical model of RC square Columns [24]. The specimen that is post heated non-jacketed (S3) is used as reference verification specimen. The detailed geometry, reinforcement is displayed in Figure 1. The load vs. deformation relationship developed experimentally that is used as reference is shown in Figure 2.

![Figure 1. Reinforcement arrangement in square column](image1)

![Figure 2. Axial strain of post heated/non-jacketed columns [24]](image2)

![Figure 3. Steps followed during Simulation of heat damaged RC Square Columns](image3)
4. Numerical Modeling

Finite element models (FEM) for Post heated unconfined RC Square column has been evolved by the usage of ABAQUS as shown in Figure 4. Concrete is defined as C3D8R which means eight noded brick element with reduced integration. Due to reduced integration, the locking phenomena observed in C3D8 element don’t show. Stress, strains are most accurate in the integration points. The integration of C3D8R element is located in the middle of element. Longitudinal and Transverse steel is defined as T3D2, two-noded 3D truss elements. Top of the concrete column takes load from steel plate so interactions defined are bottom of steel plate is declared as master surface and top of concrete column is defined as slave surface. However, these interactions are opposite at bottom of column because in that case bottom of column transfer force to steel plate. Steel is embedded in concrete.

The form of element selected and the interactions among numerous parts assembled is given in Table 1. The interaction of steel with concrete that limits the nodes of steel bars components to the compatible levels of freedom of the host neighborhood elements (concrete) is defined through embedded region constraint given in ABAQUS general. Static monotonic loading was implemented on the pinnacle with the assistance of displacement manage technique to work out the axial load-deflection records of concrete columns up to failure. A precipitated displacement of 25mm became implemented as uniformly distributed load on pinnacle of concentric columns.” Tie constraint” is used for steel plates that are actually placed at the top and bottom of column.

The parameters required to define the plasticity model of concrete are dilation angle (ψ), the plastic potential eccentricity of concrete (ε), the ratio of compressive stress in the biaxial state to the compressive stress in the uniaxial state (σb0/σc0), the shape factor of yielding surface in the deviatoric plane (Kc) and viscosity parameter. The values of all these parameters were obtained from calibration. These plates had been thought-about as rigid components with young’s modulus of 210 GPa and density of 7.85x10-9 to n/mm 3. The stress vs stress Curve for compression (Eurocode 2) and Nayal and Rasheed tension stiffening model of Concrete (2006) [25], changed for 500°C is used as input for heated unconfined concrete as shown in Figures 5(a) and 5(b) respectively. The properties of concrete that were finalized during calibration of model are given in Table 2. The seeding/Mesh size selected is 20 mm during calibration of model.

Dilation angle is a material parameter and physically, it is interpreted as an internal friction angle of concrete. The Kc (Shape factor) with a value of 0.667 is best suited for the plastic behaviour of concrete recommended by the CDP model.

<table>
<thead>
<tr>
<th>Parts</th>
<th>Element Mesh Type Chosen</th>
<th>Interactions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Column</td>
<td>C3D8R</td>
<td>Top of Concrete Column as Slave surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bottom of concrete column as Master Surface</td>
</tr>
<tr>
<td>Longitudinal steel</td>
<td>T3D2</td>
<td>Embedded in concrete</td>
</tr>
<tr>
<td>Transverse steel</td>
<td>T3D2</td>
<td>Embedded in concrete</td>
</tr>
<tr>
<td>Steel Plate</td>
<td>C3D8R</td>
<td>Bottom of Top plate as Master surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Top of Bottom plate as slave surface</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
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<tr>
<td>Poisson’s ratio, v</td>
<td>0.2</td>
</tr>
<tr>
<td>Dilation Angle</td>
<td>35</td>
</tr>
<tr>
<td>Concrete cover (mm)</td>
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</tr>
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<td>Initial and maximum increment size of the loading</td>
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</tr>
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<td>Minimum increment size Euc</td>
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</tr>
<tr>
<td>Euc</td>
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</tr>
<tr>
<td>f0/fc</td>
<td>1.16</td>
</tr>
<tr>
<td>k</td>
<td>0.67</td>
</tr>
<tr>
<td>Viscosity Parameter</td>
<td>1*10^-5</td>
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</tbody>
</table>
Figure 4. (a) Solid homogenous section; (b) Reinforcement; (c) Reinforcement embedded in concrete; (d) FE mesh; (e) Boundary conditions

Figure 5. (a) Stress vs Strain curve for compression 500 °C; Eurocode Code 2; (b) Nayal and Rasheed tension stiffening model of Concrete (2006), modified for 500 °C

4.1. Simulation of Reinforcement

The elastic behaviour of steel of steel is defined as given in Table 3. The nonlinear behavior is simulated by the use of a strain hardening ratio of zero.01 as encouraged by Kachlakev et al. (2018) [26] as shown in Figure 6.

Table 3. Elastic properties of Steel used as input

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>7.85E-009 (ton/mm³)</td>
</tr>
<tr>
<td>Young's Modulus</td>
<td>210000 (GPa)</td>
</tr>
<tr>
<td>Poison’s ratio</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Figure 6. Bilinear stress-strain behaviour of steel bars
5. Parametric Study

In order to investigate the behavior of viscosity parameter on numerical simulation of post heated RC square Column, a parametric study is performed. For this purpose 7 different models are created, tabulated in Table 3. Results are compared in the form of load deformation Curves.

6. Results and Discussions

The predictions made by Abaqus Model are very close to that of experimental. S3M3 results showed that best fit model is developed with difference of only 2.43% of experimental and modelled values. The overestimated values are represented by negative sign under the percentage diff. column in Table 4. Excellent predictions are made by models except those for higher values of viscosity parameter for Post heated 500 °C unconfined RC columns. The Principle strains shown in Figure 7 are for model S3M3 that showed best fit curve for load vs deformation plotting. The values of strain are more for heated concrete than that of controlled specimen’s i.e undamaged concrete. Maximum stresses are recorded at mid principle axis reported as 0.0015 shown by red graphics in figure.

Parametric study showed that the total number of iterations to finish the FE analysis and percentage of convergence according to step time are specified in Table 3. Moreover, ultimate load values of the test and numerical models are given, and error of numerical results in Load (KN) is compared with the test result in the Table 4. It can be clearly seen from Figure 9 that viscosity parameter plays very important role on numerical results in a way that it changes significantly the numerical load-displacement behavior of heat damaged RC columns. However load-displacement graphs could not be obtained for models S3M1 and S3M2 because the FE models did not converged. The FE model S3M1 aborted with very small percentage of convergence (6%) under value of viscosity parameter, zero which is a default value of ABAQUS software. Moreover with the definition of a very small viscosity parameter to the FE model, S3M2, the simulation similarly did not converged but the percentage of convergence has slightly increased (15%). Due to no convergent results, duration of analysis (total number of iteration) could not be measured for that FE model.

![Figure 7](image1.png)  
**Figure 7.** (i) Mid Principle, PE, (ii) Minimum Principle, PE (iii) Maximum principle, PE

![Figure 8](image2.png)  
**Figure 8.** (a) Plastic strain Magnitude, (b) Translation displacement, (c) Active Yield (AC Yield)
With the increase in value of $\mu$, numerical models have started to converge. For the models, S3M3 and S3M4, the numerical results are very similar to that of experimental in terms of load-displacement behavior of the tested damaged RC columns. Percentage of error in ultimate load level stayed under 5% as well. Total number of iterations for S3M3 and S3M4 are 1211 and 1048 respectively. Numerical load-deformation behaviors of the models of S3M5 through S3M7 have started to lose their fitness due to increase in value of viscosity parameter (above 0.0005). When the value of $\mu$ is above 0.005, the models of S3M6 and S3M7 showed very weak behavior and the results substantially deviated from that of experimental. However total number of iterations decreased significantly. Plastic strain magnitude is abbreviated as, PEMAG. For most of the material this magnitude is equal to Equivalent plastic strain. Figure 8(a) shows the values of plastic strain magnitude. PEMAG is maximum at the centre of column and reduces up to the top and bottom. UT, that represent all translation displacement components is shown in Figure 8(b). Active yield (AC Yield) is an important parameter showing that plastic flow has taken place in simulation after application of load. The value of AC Yield 1 confirms plastic flow while AC value 0 shows no plastic flow region as shown in Figure 8(c).

![Figure 8(a): Load-deformation behaviors of models S3M3 and S3M4](image)

![Figure 8(b): Active yield (AC Yield)](image)

![Figure 8(c): Plastic strain magnitude PEMAG](image)

**Figure 8.** Load-deformation behaviors of models S3M3 and S3M4, Active yield (AC Yield) and Plastic strain magnitude PEMAG

**Table 4. Results of the experimental and numerical study**

<table>
<thead>
<tr>
<th>Name of Model</th>
<th>$\mu$</th>
<th>Total iterations</th>
<th>Convergence (%)</th>
<th>Load (KN)</th>
<th>Diff (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>864</td>
<td>-</td>
</tr>
<tr>
<td>S3M1</td>
<td>0</td>
<td>n/a</td>
<td>6</td>
<td>n/a</td>
<td>n/a</td>
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<tr>
<td>S3M2</td>
<td>0.00001</td>
<td>n/a</td>
<td>15</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
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<td>0.00005</td>
<td>1211</td>
<td>100</td>
<td>843</td>
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<tr>
<td>S3M4</td>
<td>0.0001</td>
<td>1048</td>
<td>100</td>
<td>893</td>
<td>-3.35648</td>
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<tr>
<td>S3M5</td>
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<td>100</td>
<td>915</td>
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<td>S3M6</td>
<td>0.001</td>
<td>848</td>
<td>100</td>
<td>933</td>
<td>-7.98</td>
</tr>
<tr>
<td>S3M7</td>
<td>0.01</td>
<td>456</td>
<td>100</td>
<td>1211</td>
<td>-40.1</td>
</tr>
</tbody>
</table>

**7. Conclusion**

The numerical verification of existing experimental study was carried out in order to investigate the sensitivity of viscosity parameter. The results were compared in terms of load displacement relationship, time and rate of convergence. Following conclusions are drawn from the study performed.

- Optimum value of viscosity parameter that reduces time and increases convergence should be selected.
- The optimum value of 0.00005 or 0.0001 should be selected as it gives excellent fit model for heated damaged unjacketed RC Square Columns.
- Above 0.0005 the values divert a lot so its use is discouraged though it reduces time and increments.
- Study should be conducted with varied values of the parameter in order to improve calculation accuracy of numerical simulation of post heated unconfined RC square columns.
8. Funding

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9. Conflict of Interest

The authors declare no conflict of interest.

10. References

[1] Reported by ACI Committee 440, "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures", American Concrete Institute, 2008.


Increasing the Contribution of GFRP Bars on the Compressive Strength of Concrete Columns with Circular Cross Section

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Abstract

Corrosion of steel in concrete elements is a major issue in concrete structures. In order to overcome this matter, Glass Fiber Reinforced Polymer (GFRP) reinforcement is being used in concrete members from almost 20 years ago. Although it has been used and developed in recent years, there are still some uncertainties for the application of FRP reinforcement, especially in concrete columns. Most codes such as ACI, CSA, JSCE & etc. neglects the effect of these reinforcements or they do not permit them in compressive concrete elements. In this essay, it has been shown that these rebar can contribute significantly in compressive strength of concrete columns if the column confinement is provided sufficiently. In order to achieve the required confinement to reach a sharp contribution of GFRP longitudinal rebar in concrete columns, the spiral of FRP rebar with small pitches around longitudinal rebar is taken into account. This leads to higher strains of concrete which can result in a higher contribution of FRP longitudinal rebar. Foremost, equations related to the compressive strength of concrete columns considering the influence of spiral confinement will be carried out. Then, a parametric study will be performed, and the effects of pitch, concrete strength, column diameter, the quantity of longitudinal rebar and concrete cover will be investigated.

Keywords: GFRP Rebar; Confinement; Circular Concrete Column; FEM; Parametric Study.

1. Introduction

Reinforced concrete columns, as members of a structure which carry loads of structures, mainly compressive, are the most important members in terms of performance and the stability of concrete structures. Formerly, these columns were reinforced, only with steel reinforcements. But if these columns were placed in corrosive environments such as marine environments, the result would be the corrosion of steel reinforcements that eventually can destroy the structure. In general, corrosion of steel reinforcement is the most important factor in determining the life expectancy of concrete structures. The process of repairing structures is one of the most costly matters; therefore, in some cases, approximately, the repair and renovation of a concrete structure can cost twice the initial construction price of the structure. In Canada and the United States, billions of dollars are spent annually on the repair, renovation, and maintenance of bridges and marine structures. Governments and industrial units are looking for suitable infrastructure systems to provide more durable and more corrosion-resistant structures in order to allocate fewer funds for maintenance and repair of these structures. Regarding this, engineers around the world are looking for new and cost-effective products along with innovative approaches and systems to solve these issues. As a result, in order to meet these objectives and improve the

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performance of concrete structures, a replacement was introduced as steel rebar for the reinforcement of concrete, called FRP reinforcement.

The Glass Fiber Reinforced Polymer (GFRP) rebar is the most commonly used rebar among FRP reinforcement, mainly due to its low price in comparison with other types of FRP rebar, such as CFRP and AFRP. The contribution of GFRP reinforcement in compressive strength of columns is still unclear and requires supplementary evidence to endorse its performance. At the ultimate concrete crushing strain (around 0.003), the GFRP rebar can attain only a small portion of its ultimate strength, due to its higher ultimate compressive strain (around 0.01) and its linear behaviour. In this paper, it has been tried to increase the ultimate concrete crushing strain by increasing the confinement of concrete, in order to enhance the contribution of GFRP rebar in compressive strength of columns. This could lead counting on GFRP longitudinal rebar strength on compressive behaviour of concrete columns, in spite of neglecting them, which is currently recommended by many design codes.

1. Background

In the past few years, a lot of research has been done on the use of FRP composites in structures. Composite FRP materials are used as internal and external reinforcing elements of concrete. The use of FRP reinforcement in concrete columns still faces uncertainties, therefore the use of this type of rebar in the report of the 440.1R Committee of ACI in 2006 [1] in columns and compression members due to lack of sufficient research is prohibited. However, by doing some research over the past years on the behaviour of concrete pressure members with FRP reinforcement, the use of these reinforcements in newer regulations, such as the report no. 40 of “fib” namely, FRP Reinforcement in RC Structures [2] and the ACI Committee's 440.1R report in 2015, these reports still have not allowed counting on the effect of FRP reinforcements on compression elements, and they neglect these rebar. The reason for this can be cited in three factors: 1: There is a few number of research in this field. 2: Low resistance and deficiency of this type of reinforcement in compression. 3: The lower contribution of this reinforcement to compression due to the low modulus of compression elasticity of these rebar, compared to steel reinforcements. In this research, we are going to increase the contribution proportion of FRP rebar in compressive concrete members with a circular section using the confinement effect of FRP rebar spirals.

Heretofore, little research has been done on the behaviour of reinforced concrete columns with FRP reinforcements. In 2012, Tobbi et al. carried out experiments on eight concrete square columns of 350 mm dimension, reinforced with steel rebar and GFRP, and examined the effects of the arrangement and spacing of transverse rebar. All samples had a constant ratio of longitudinal rebar (1.9%). Two of the samples had steel rebar, and five had a GFRP rebar, and there was a sample without any reinforcement. Figure 1 shows the dimensions and properties of Tobbi’s tests. By analysing and comparing results with the ACI and CSA Canadian codes, the researchers (Tobbi et al. 2012) reached the following results [3]:

- The early crushing of concrete cover reduces the axial capacity before any enclosing effect occurs. After the concrete cover was completely crushed, there was a significant increase in strength, ductility, and stiffness for the core of the concrete in highly confined specimens. These observations suggest that only the effect of the concrete core can be used in the compression calculations of the columns unless special arrangements are made to prevent the segregation of the concrete cover.

- Examining the type of cross-sectional reinforcement and their spacing showed the positive effect of GFRP reinforcement as transverse reinforcement in increasing the strength, ductility, and stiffness of the concrete core. It was also shown that if the spacing of the transverse reinforcements declines, the effect of the enclosure will increase and it will prevent the buckling of the longitudinal reinforcements. In these experiments, reducing the spacing of transverse reinforcements from 120 mm to 80 mm increased the axial capacity of the column by more than 20%.

- The 0.85 reduction factor, mentioned in concrete columns for steel reinforcement accepted by different regulations can also be used for GFRP reinforcements.

- GFRP reinforcements have about 10% proportion of the whole load axial strength of the column, which is approximately equal to the proportion of steel reinforcements (about 12%). This indicates that GFRP reinforcements have an effective contribution to pressure, provided they are properly enclosed to prevent buckling of reinforcement [3].
In 2013, Afifi studied the experimental study of the axial behavior of concrete columns with a circular cross-section, reinforced with GFRP and CFRP reinforcements. He tested a total of 27 circular concrete columns of 300 millimeters diameter. He categorized his samples into three groups. The first group consists of three samples, one of which is a simple concrete (without any reinforcement), and two other with steel reinforcements. The second group consists of 12 concrete columns reinforced with the GFRP longitudinal and transverse reinforcement, and the third group is identical to the second group, except that the type of reinforcement has changed from GFRP to CFRP. In these experiments, various parameters such as the type of reinforcement, the longitudinal reinforcement ratio, the volumetric spiral reinforcement ratio, the diameter of the reinforcement, the distance between the spirals, the arrangement of the transverse reinforcements, and the length of the ties’ overlap were investigated. Figure 2 shows the overall view of the samples tested and some of its properties [4].

Some of the results of experiments conducted by Afifi (2013) can be summarized as follows:

- FRP reinforcements are effective in tolerating compressive axial stresses even after the crushing of concrete. GFRP reinforcements make up 8% of concrete capacity on average. This value is 13% for CFRP reinforcements and 15% for steel reinforcements.
The reinforced samples of the GFRP rebar have an average capacity of 7% lower than the reinforced samples to the steel rebar, while the columns with CFRP rebar have an average capacity of 5% lower in comparison with the same columns reinforced with steel rebar.

The use of GFRP and CFRP rebar as transverse reinforcements, according to the provisions of CSA S806-12, provides sufficient buckling restraint against longitudinal reinforcements and provides proper confinement in concrete at post-peak stages.

The study concludes that neglecting the effect of longitudinal FRP reinforcements in Canadian code (CAN/CSA S806-12) underestimates the capacity of concrete column capacity.

The GFRP and CFRP rebar in these experiments reached up to 75% of their ultimate compressive strain, indicating the effectiveness of these rebar in bearing pressures, even after the failure of concrete [4].

Hadhood et al. (2017) and Salah-Eldin et al. (2019) performed some experimental tests on circular and square high-strength concrete columns with GFRP longitudinal and transverse rebar. They concluded that GFRP rebar can contribute on concrete strength and stiffness, similar to steel-reinforced columns in some cases [5, 6].

2. Tensile Behaviour of FRP Rebar

There is no plastic behaviour, even up to failure in FRP reinforcements, when a tensile force is applied to them, and they show a totally elastic behaviour. Consequently, there is no specific yield point for these rebar.

3. Compressive Behaviour of FRP Rebar

The experiments performed by Wu (1990) on the compressive behaviour of FRP reinforcement with a low ratio of length to diameter showed that the compressive strength of this reinforcement was less than its tensile strength. The compressive strength of FRP reinforcements for all types of GFRP, CFRP, and AFRP has been reported to be 55, 78 and 20%, respectively, of their tensile strength. Also, similar results are obtained for the modulus of compression elasticity of these types of reinforcements. According to the research, experiments, and reports, the modulus of elasticity of the FRP reinforcement in compression for the GFRP, CFRP and AFRP types is reported to be 80, 85 and 100%, respectively, of the elasticity modulus in tension [7, 8].

4. Confined Concrete Behaviour

The behaviour of the confined concrete was first investigated experimentally in 1928 by Richart et al. The purpose of this experiment was to determine the effect of stress tolerance of concrete in one direction, in the presence of stress in other directions. They concluded that the presence of confinement stresses in the lateral directions, can increase the strength of the concrete in the longitudinal direction, and in a constant quantity of lateral stresses, this increase was approximately constant and did not correlate with the concrete mixtures. In Figure 3, the effect of the lateral confinement pressure on the stress-strain diagram of concrete is shown [9].

![Figure 3. The effect of confinement pressure on stress-strain behaviour of concrete [9]](image-url)
Mander et al. (1988) developed a stress-strain model for concrete elements under uniaxial compressive stress and confined by transverse rebar. They proposed equations for predicting the behaviour of concrete members confined by transverse rebar such as spirals or ties. These equations are used in this research to predict the behaviour of concrete columns reinforced with GFRP rebar. Figure 4 shows the proposed stress-strain model of confined and unconfined concrete [10].

According to Mander et al. (1988), the confined compressive strength of the concrete column can be calculated by Equation 1 [10].

\[
f'_{cc} = f'_{co}(-1.254 + 2.254 \sqrt{1 + \frac{7.94f'_{fl}}{f'_{co}} - 2 \frac{f'_{fl}}{f'_{co}}})
\]

(1)

Where \( f'_{fl} \) is the effective lateral confining stress from spirals given by Equation 2:

\[
f'_{fl} = f_l k_e
\]

(2)

Where \( f_l \) is the lateral pressure from the transverse reinforcement and \( k_e \) is confinement effectiveness coefficient. These parameters for circular cross-sections are given by Equations 3 and 4, respectively.

\[
f_l = \frac{1}{2} \rho_s f_{yh} = \frac{1}{2} \left( \frac{4A_{sp}}{d_s} \right) f_{yh} = \frac{2A_{sp}f_{yh}}{d_s}
\]

(3)

\[
k_e = \frac{A_e}{A_{cc}} = \frac{\pi(d_s - \frac{r'}{2})^2}{4d_s^2(1 - \rho_{cc})} = \frac{\pi d_s^2(1 - \frac{r'}{2d_s})^2}{4d_s^2(1 - \rho_{cc})} = \left( \frac{1 - \frac{r'}{2d_s}}{1 - \rho_{cc}} \right)^2
\]

(4)

In Equations 3 and 4, the definition of used parameters are as below and also shown in Figure 5:

- \( \rho_s \): Ratio of the volume of transverse confining steel to the volume of confined concrete core;
- \( f_{yh} \): Yield strength of the transverse rebar;
- \( A_{sp} \): Area of transverse rebar;
- \( s \): Center to center spacing of spiral rebar;
- \( A_e \): The area of an effectively confined concrete core at midway between the levels of transverse reinforcement;
- \( A_{cc} \): The area of concrete core.

Also, the maximum confined concrete strain can be calculated according to Equation 5 [10].

\[
\varepsilon_{cc} = \varepsilon_{co}[1 + 5 \left( \frac{f'_{cc}}{f'_{co}} - 1 \right)]
\]

(5)
5. Parametric Analysis

In this research, the effect of spirals on the contribution of longitudinal GFRP rebar will be investigated on circular concrete columns based on Equations 1 to 5. The GFRP rebar material properties are considered as shown in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\alpha = \frac{E_{fuc}}{E_{fat}}$</th>
<th>$\beta = \frac{f_{fuc}}{f_{fat}}$</th>
<th>$f_{fat}$ (MPa)</th>
<th>$\varepsilon_{fat}$</th>
<th>$E_{fat}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP rebar</td>
<td>0.8</td>
<td>0.55</td>
<td>627</td>
<td>0.014</td>
<td>44785</td>
</tr>
</tbody>
</table>

In Table 1, $E_{fuc}$ and $E_{fat}$ are the compressive and tensile elasticity modulus of GFRP rebar, respectively. Also, $f_{fuc}$ and $f_{fat}$ are the compressive and tensile strengths of GFRP rebar, respectively. The parameter $\varepsilon_{fat}$ is referred to the tensile ultimate strain of GFRP rebar. The parameters that are considered in this research are spiral pitch (center-to-center spacing of spirals), concrete strength, column diameter, the quantity of longitudinal rebar and the clear cover of concrete.

The contribution percentage of GFRP rebar in compressive strength of circular concrete columns (“CP” parameter) is calculated based on Equation 6.

$$CP = \frac{P_1 - P_0}{P_0} \times 100 \quad (6)$$

Where $P_0$ is the ultimate compressive strength of concrete column without any reinforcement (for the sole concrete) and is calculated according to Equation 7.

$$P_0 = 0.85 \times f'_{co} \times \left(\frac{\pi}{4} D^2\right) \quad (7)$$

Also, $P_1$ is the ultimate compressive strength of concrete column with the GFRP rebar and with the effect of confinement of spirals. $P_1$ is calculated according to Equation 8.

$$P_1 = 0.85 \times f'_{cc} \times \left[\frac{\pi}{4} \left( d_s - \frac{s'}{2} \right)^2 \right] + \min\{\varepsilon_{cc} E_{fuc}, f_{fuc}\} \times A_{GFRP} + 0.85 \times f'_{co} \times \left[\frac{\pi}{4} D^2 - \left( d_s - \frac{s'}{2} \right)^2 \right] \quad (8)$$

The procedure of calculating the contribution percentage of GFRP rebar in compressive strength of circular concrete columns, and how confinement can increase it, is shown in Figure 6.
Figure 6. The flowchart for calculating the contribution percentage of GFRP rebar in compressive strength of circular concrete columns

The base sample model is considered as a 500 mm diameter circular column with 50 mm concrete cover (from the outermost edge of concrete to the center of spiral rebar). The diameter of the GFRP spiral is considered 8 mm with a pitch spacing of 100 mm and a total number of 6 longitudinal GFRP rebar with 25 mm diameter is considered. The concrete strength for the base model is also assumed 35 MPa. For the parametric study, the parameters are changed with respect to this base model. For instance, for investigating the effect of spiral pitch spacing on the contribution percentage of GFRP longitudinal rebar in compressive strength of the column, this parameter (spiral pitch spacing) is altered from 50 mm to 250 mm, and other parameters have remained unchanged with respect to the base model.

Figure 7 shows the influence of spiral pitch and concrete strength on contribution percentage (CP) of GFRP longitudinal rebar in the compressive strength of the concrete column. According to this figure, spiral pitch spacing has a considerable effect on the contribution of GFRP rebar. By increasing the spiral pitch spacing from 50 mm to 250 mm, the GFRP rebar contribution percentage decreased from 37.8% to 5.0%. Reducing the spiral pitch spacing increases the confinement of the concrete column, which leads to raising the percentage of contribution of longitudinal rebar because these rebar can experience higher strains and consequently higher stresses and forces. Moreover, according to Figure 7, it can be concluded that increasing the concrete strength ($f'_c$) will slightly reduce the contribution of GFRP longitudinal rebar in the axial capacity of columns. This is because as the $f'_c$ increases, the confinement effect will be limited.
Figure 7. Spiral Pitch and Concrete Strength Effect on Contribution Percentage of GFRP Longitudinal Rebar in Compressive Strength of Column

Figure 8 shows the effect of concrete cross-section diameter on contribution percentage (CP) of GFRP longitudinal rebar in the compressive strength of the concrete column. According to this figure, the diameter of the column has a substantial influence on the contribution of GFRP rebar, such that the more dimension of column diameter, the less the contribution percentage of GFRP longitudinal rebar. By increasing column diameter from 250 mm to 1000 mm, the GFRP rebar contribution percentage reduced from 43.0% to 9.8%.

Figure 9 shows the effect of quantity of longitudinal GFRP rebar on contribution percentage (CP) of GFRP longitudinal rebar in the compressive strength of the concrete column. According to this figure, as the quantity of longitudinal GFRP rebar increases, the contribution of GFRP rebar will be increased gradually and linearly. Therefore, increasing the longitudinal rebar from 1000 mm² (which is almost equivalent to 0.5% of column gross section) to 4000 mm² (which is almost equivalent to 2.0% of column gross section) can alter the CP factor from about 13.4% to 19.7%. The reason for this is increasing in the area of longitudinal rebar in comparison with the gross cross-sectional area of the column, which yields a higher contribution.
Figure 9. Quantity of Longitudinal Rebar Effect on Contribution Percentage of GFRP Longitudinal Rebar in Compressive Strength of Column

Figure 10 demonstrates the effect of concrete cover dimension on contribution percentage (CP) of GFRP longitudinal rebar in the compressive strength of the concrete column. According to this figure, the concrete cover parameter changed from 25 mm to 100 mm which is a typical range among members in various real structures. The contribution of GFRP rebar will be declined gradually and almost linearly as the concrete cover increased. However, the change in the CP factor is about 5.4% (from about 19.4% to 14.0%) as concrete cover rise from 25 mm to 100 mm. This is because of the reduction in the confinement area of the concrete column as the concrete cover increases, which yields a lower contribution.

Figure 10. Concrete Cover Effect on Contribution Percentage of GFRP Longitudinal Rebar in Compressive Strength of Column

Figure 11 shows the effect of transverse rebar diameter on contribution percentage (CP) of GFRP longitudinal rebar in the compressive strength of the concrete column. According to this figure, as the size (diameter) of transverse rebar increased from 6 mm to 14 mm, the contribution of GFRP rebar will be increased significantly. This is because of the confinement effect of spirals that will increase with a larger diameter (size) of spirals.
Figure 11. Size of Transverse Rebar on Contribution Percentage of GFRP Longitudinal Rebar in Compressive Strength of Column

6. Verification

In order to verify the results in the previous section, firstly, a numerical Finite Element Method (FEM) model using ABAQUS software was created based on Afifi’s (2013) experimental test, and the results are verified. This step was performed to verify the results of the ABAQUS software. Afterward, another numerical finite element model based on the base model as defined previously was created to verify the results given in the previous section of this research.

In the ABAQUS model, the loading is applied through a rigid plate in the top of the column, monotonically, in order to avoid stress concentration on the main column and distribute the compressive force on the total area of the column. The mesh elements used in the numerical model is of 3-dimensional 8-node hexahedral elements with reduced integration (C3D8R) for concrete parts and 3-dimensional 2-node truss with reduced integration (T3D2) for GFRP rebar parts. Amiri et al. showed that these elements are proper for non-linear static and dynamic analysis and they follow the constitutive law of integration [11]. Also, the concrete material properties is defined in the software, using concrete damaged plasticity model, with dilation angle ($\psi$) of 40°, shape factor for yield surface (K) of 0.72 and ratio of initial equi-biaxial compressive yield stress to initial uniaxial compressive yield stress ($f_{0y}/f_{c''}$) of 1.15 [12-16].

For the first stage, the model named “G8V-3H80” by Afifi (2013) is used. This model is a 300 mm diameter circular column with 8 No. 15.9 mm longitudinal GFRP rebar, and spiral No. 9.5 mm with a pitch of 80 mm. Figure 12 shows the 3D-model, which is created exactly the same as Afifi’s (2013) experimental test model named “G8V-3H80”.

Figure 12. Overall View of Created Model in ABAQUS Software Based on Afifi’s (2013) “G8V-3H80” Experimental Sample

Figure 13 compares the Load-Strain curve of the analyzed FEM model and the curve obtained from Afifi’s (2013) experimental test named “G8V-3H80”. According to this figure, it can be seen that the results are very close, especially from the starting point of the curve to the ultimate load level. However, in the post-peak region, there is a modest difference which is not significant for this research.
Figure 13. Comparison of the Load-Axial Strain Curves of Afifi’s (2013) experimental Test versus FEM

Figure 14 demonstrates the ultimate strain of GFRP longitudinal rebar and spiral in the FEM model which is created based on Afifi’s (2013) “G8V-3H80” Experimental Sample.

Figure 14. Ultimate Strain of GFRP Spiral and Longitudinal Rebar in the FEM Model Similar to Afifi’s (2013) “G8V-3H80” Experimental Sample

Figure 15 shows the maximum principal plastic strain of concrete in the FEM model which is created based on Afifi’s (2013) “G8V-3H80” Experimental Sample.

Figure 15. Maximum principal plastic strain of concrete in the FEM Model Similar to Afifi’s (2013) “G8V-3H80” Experimental Sample
In the second stage of verification, another FEM model is created to verify the results which were obtained in the earlier stage for the base model. The load-axial strain for this model is shown in Figure 16. According to this figure, the ultimate load of FEM model is about 7035 KN which is almost 2.5% higher than the calculated axial capacity based on the method presented in the previous section, which is a negligible error.

Figure 16. Load-Axial Strain Curve of FEM Model Similar to Base Model

2. Conclusions

In this research, the effect of parameters such as spiral pitch, size of spiral, concrete strength, concrete cover, column diameter, and longitudinal rebar area are investigated on the confinement and therefore increase of compressive axial capacity of the concrete column reinforced transversally and longitudinally with GFRP rebar. In contrary with various codes and standards such as ACI, CSA and JSCE that neglects the contribution of longitudinal GFRP rebar in compressive strength of concrete columns, it has been shown that by increasing the confinement of the concrete column (for instance by decreasing spiral pitch or concrete cover) the contribution percentage of longitudinal rebar in axial compressive strength can be enhanced, even up to 45% in the considered samples. Briefly, the following results can be concluded:

- Spiral pitch, spiral size (diameter) and column diameter are the most influential parameters affecting the contribution percentage of GFRP rebar on compressive axial strength of circular concrete columns.
- Decreasing spiral pitch from 250 mm to 50 mm, increased the contribution percentage (CP) by 32.8% (from 5.0% to 37.8%).
- Increasing spiral diameter size from 6 mm to 14 mm increased the CP parameter by 33.1% (from 11.01% to 44.1%).
- Declining the column diameter from 1000 mm to 250 mm, increased the CP parameter by 33.2% (from 9.8% to 43.0%).
- The parameters, concrete strength, longitudinal rebar area, and concrete cover do not change the CP parameter significantly.
- Declining the concrete cover from 100 mm to 25 mm, increased the CP parameter by 5.4%.
- Declining the concrete strength from 40 MPa to 30 MPa, increased the CP parameter by 6.4%.
- Increasing longitudinal rebar area from 1000 mm² (equivalent to 0.5% of column gross section) to 4000 mm² (equivalent to 2.0% of column gross section), increases the CP parameter by 6.3%.

To conclude, the longitudinal GFRP rebar can considerably contribute to the compressive strength of concrete columns by increasing the confinement of the column. In a specific column diameter, decreasing spiral pitch and increasing spiral size are the most noticeable parameters in increasing the contribution. Subsequently, design codes’ assumption of neglecting the effect of GFRP compressive rebar is conservative and it could be waived, if the confinement effects would be taken into account.
3. Conflicts of Interest
The authors declare no conflict of interest.

4. References


