Evaluating the Damage Content of Karbandi Using Frequency Domain Analysis (Case Study: Timche Haj-Mohammad-Qoli of Tabriz Historic Bazaar)

Elshan Ahani  
Shanghai Jiao Tong University

Ali Ahani (ahani16@itu.edu.tr)  
Istanbul Technical University

Research Article

Keywords: Tabriz bazaar complex, Karbandi, Structural morphology, Fourier analysis, Damage content, Near-field ground motion.

Posted Date: August 8th, 2022

DOI: https://doi.org/10.21203/rs.3.rs-1013551/v2

License: This work is licensed under a Creative Commons Attribution 4.0 International License.
Read Full License
Evaluating the Damage Content of Karbandi Using Frequency Domain Analysis (Case Study: Timche Haj-Mohammad-Qoli of Tabriz Historic Bazaar)

Elshan Ahani\textsuperscript{1a}, Ali Ahani\textsuperscript{2b}\textsuperscript{*}

\textsuperscript{1} Ph.D. Candidate of Structural Engineering, Shanghai Jiao Tong University

\textsuperscript{2} M.Sc. graduate of Structural engineering, Istanbul Technical University

Abstract

In this study, an alternate method to evaluate structural systems, especially those with historical importance has introduced by applying Fourier Transform (FT) to the damage ratio of time history outcome in the frequency domain. The concept of damage content (DC) regarding the mechanical characteristics of the used material, including plastic strain, failure plane, and ultimate load-bearing capacity, along with drift value, record selection criteria, and architectural aspects, have employed. Due to its valuable aesthetic and architectural view, Timche Haj-Mohammad-Qoli of Tabriz Historic Bazaar, one of the traditional covered spaces with the complicated configuration of spatial masonry intersecting arches, selected for further assessment in the current research program. The required experimental samples for obtaining the mechanical properties and relevant geometrical measurements to prepare the numerical model of the structure obtained. The strong ground motions according to seismological and geological characteristics of the construction site selected. The records with different durations were merged by Fourier Transform (FT) and Damage Content (DC) analysis. According to the outcomes, the damage state of the structure due to the imposed strong motion at every stage of the lateral loading from the failure initiation to the final collapse was traceable. As a straightforward outcome, the vulnerability of the near-

\textsuperscript{a} Ph.D. Candidate of Structural Engineering, Shanghai Jiao Tong University, Shanghai 200240, China, Email: e.ahani@sjtu.edu.cn, \url{https://orcid.org/0000-0002-8483-6547}

\textsuperscript{b} M.Sc. graduate of Structural engineering, Istanbul Technical University, Istanbul 34469, Turkey (corresponding author), Email: ahani16@itu.edu.tr, \url{https://orcid.org/0000-0002-1447-7171}
field earthquakes is more tangible than that of the far-field earthquakes. The provided methodology could have proper use in future similar studies for evaluating the performance of the structures.

**Keywords:** Tabriz bazaar complex, Karbandi, Structural morphology, Fourier analysis, Damage content, Near-field ground motion.

### 1 Introduction

Several historical and ancient masonry buildings are belonging to the vast circle of arches and vaults. Aesthetic, slenderness, and load-bearing capability make these structures attractive for manipulation in the construction industry of monumental structures both in past and present (Sarhangi, 1999; Sevim et al., 2011; Sadeghi et al., 2019). However, numerous vulnerability points of these structures, to a large extent because of the limited load-bearing capacity of the masonry elements, make them laid open to natural hazards (D'Ayala and Speranza, 2003; Lourenço and Roque, 2006; Pasticier et al., 2008; Asteris et al., 2014; Asteris et al., 2019). Severe damages and failures caused by earthquakes demonstrate that masonry structures, including masonry arches and vaults on the top, are one of the most vulnerable elements in historical monuments (Lagomarsino and Podestà, 2004; Dogangun and Sezen, 2012; Formisano and Marzo, 2017; Cusano et al., 2019). Iran is a host country for numerous masonry structures that demonstrated the vulnerability of masonry arches to strong ground motions in numerous earthquakes consist of but not limited to the Tabas earthquake of 1978, the Manjil earthquake of 1990, the Ahar-Varzeghan earthquake of 2012, and the Sarpol-e Zahab earthquake of 2018.

The typologies of the heritage buildings are usually capable of resisting the gravity loads. However, they are weak to resist the seismic forces, and in some cases, vehemently ruined due to severe earthquakes. As a result, the seismic events gave a strong motivation towards understanding and interpreting the seismic response of masonry structures to different structural typologies (Bell, 1903; IASS, 1984; Nooshin, 1998), especially those exposed to high seismic hazards or unfavorable soil properties (Motro, 2009). Several studies devoted to analyzing historical masonry structures, such as churches, monuments, bazaars, and palaces (Haj Gasemi, 2004; Stach, 2010; Ahmadi, 2014; Kawaguchi, 2016; Aghabeigi et al., 2020).
Giuffre’ (1991; 1993) demonstrated the necessity of a multidisciplinary technique for seismic assessment of historical masonries. Binda and Saisi (2005) indicated the importance of on-site observations and experimental investigations to achieve the reliable structural information on masonry buildings, such as geometry, structural details, crack patterns, etc. In another study performed by Lagomarsino et al. (2004), the importance of modeling strategies to assess the seismic response of masonry structures utilizing macro elements instead of conventional evaluation of structural behavior suggested. Doglioni et al. (1994) used the simplified techniques of kinematic mechanism to assess the seismic response of the churches. Lourenço (2002) showed even if the definition of the parameters to model a non-linear numerical simulation subjected to difficulties in many cases, it is advantageous to compare the other modeling methods to assess the seismic behavior of historic structures. More in advance during the past decade, investigations on three-dimensional simulations of geometrically complex structures performed to evaluate the seismic behavior of historical buildings (Casarin and Modena, 2008; Boscato et al., 2014; Castellazzi et al., 2018; Mendes et al., 2020). As yet, modeling strategies using numerical techniques, such as discrete element method (DEM) and finite element method (FEM) by considering non-linear material properties becomes a standard procedure in simulation of full-scale structures (Roca, 2001; Rots, 2001; Ramos and Lourenço, 2004). Most of the historical monuments and ancient buildings made of masonry elements, including but not limited to clay bricks, stones, raw clays, etc. Advances in using numerical methods to ease the calculation process in the design of structural elements demonstrate the complexity of considering the masonry elements presence. Because of the unpredictable post-failure behavior of these elements and their utilization inside the old-fashioned buildings, the procedure becomes even more difficult than anticipations. Hence, many researchers used FEM to predict the behavioral content of masonries in historical buildings (Chiarugi et al., 1993; Croci, 1995; Roca et al., 2010; Atamturktur and Laman, 2012). The complicated geometric shape of many historical buildings constrains the modeling procedure even in FEM software, in which the meshing is affected by numerous parameters, including convergence and unity problems. To solve this, new methods of modeling named simplified micro-modeling and macro-modeling with more simple geometry were used by the researchers (Betti and Galano, 2012; Vicente et al., 2018; Ahani et al., 2019). Predicting the behavioral content of masonries due
to the material diversity and load-displacement indices confronted with sensible errors. Hence, numerous behavioral contents were introduced by researchers to provide a better perspective on the behavior of masonry elements, many of which are also approved by prominent standards and design codes (FEMA 308, 1999; Eurocode 8, 2005). This is while many design codes and standards remained silent about the behavioral content of the masonry elements (ICC, 2003; Issue No. 120, 2003; ACI530, 2011; Standard 2800, 2015). Concrete damage plasticity as an empirical method was used in many experimentally and numerically performed research studies (Wang and Hsu 2001; Majewski, 2003). The accuracy of this method compared to the other approaches like concrete smeared crack and Drucker-pruger has more appropriate convergence. Hence, this method is introduced by Eurocode 2 (2005) to anticipate the post-failure behavior of concrete elements.

Timche Haj-Mohammad-Qoli of Tabriz Historic Bazaar as a Cultural Heritage by further evaluation of its structural morphology and damage content considered the study case of this research. The basic steps for evaluating masonry buildings are to achieve adequate knowledge about the structure, materials, and the consecutive use of the obtained data for seismic evaluation purposes. To debrief, the historical and architectural description of the Tabriz bazaar and Timche Haj-Mohammad-Qoli, an on-site observation to identify the current status of the structure and its seismic assessment with the aid of Nonlinear Time History Analysis (NTHA) and Fourier Analysis (FA) have performed in the current research. The assessment of masonry structure performed in two states of subjecting to gravity loads, and gravity and seismic loads.

2 Research Significance

One of the most controversial issues in surveying historical monuments relates to the prevailing structural vulnerability that regarding the cultural significance, architectural esthetic, and consideration as national monuments may intensify the possibility of further excavations for these structures. In this regard, numerous Standards (FEMA 306, 1998; FEMA 308, 1999; ACI 228.2R-98, 2004; ACI 364.1R-07, 2007; ACI 201.1R-08, 2008; ACI 369R-11, 2011; ACI 562M-13, 2013; ASTM C597-16, 2016) introduced some inspecting methods accompanied by non-destructive tests (NDT) to acquire an assumption for the current
stance of the structures. This is while in some cases, visional inspections with the assistance of NDT tests won't provide sufficient information about the studied structures. Hence, standards (ASCE/SEI 31-03, 2003; ACI 364.1R-07, 2007; ASCE/SEI 41-06, 2007; ACI 562M-13, 2013; ASTM C496/C496M-17, 2017; ASTM C1314-18, 2018; ASTM C42/C42M-20, 2020; ASTM C39/C39M-21, 2021) also introduced destructive tests as an ultimate and irreplaceable alternative for structural evaluation of the structures. In this condition, since the facade, stability, and the integrity of the historical monuments is substantially important (FEMA 172, 1992; Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring & Reconstructing Historic Buildings, 1995; ASCE/SEI 31-03, 2003; FEMA P-774, 2009), the amount and location of the test become an important parameter in seismic assessment, which may vary according to the principles of the standards (FEMA 274, 1997; FEMA 306, 1998; ASCE/SEI 31-03, 2003, ACI 369R-11, 2011).

While limited standards provided protocols for testing and getting specimen samples from the historical structures (ASCE/SEI 31-03, 2003; ASCE/SEI 41-06, 2007), most of many (ATC-40, 1996; FEMA 306, 1998; ACI 201.1R-08, 2008) introduced the destructive tests for modern structures that were not applicable for historical masonries. Thus, the provided methods may not satisfy the required limits of historical monuments, and therefore while the destructive tests may have limited eligibilities (Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring & Reconstructing Historic Buildings, 1995; 36 CFR 68, 2012) and the outcomes of NDT tests may not comprehensively reliable for the long-lasted human heritages a thorough numerical evaluation shall be employed. In addition to the difficulty of finding the failure amount and location, determining them may of high value for the historical structures since the financial worthiness of these cultural heritages is incomparable with typical engineering structures. In this regard, many studies have evaluated the failure parameters of historical masonries (Zampieri et al., 2016; Zampieri et al., 2017; Zampieri et al., 2018; Galassi et al., 2018; Chen et al., 2019). According to carried out studies and the provided principles of the regulations and standards, the failure parameters were categorized as but not limited to soil settlements or support movements (FEMA 172, 1992; FEMA 274, 1997; Portioli and Cascini, 2017; Galassi et al., 2018; Zampieri et al., 2018), failure due to ground motion excitations (FEMA 172, 1992; Eurocode 8, 2005;
Concerning the mentioned implementation, an effective and economical method for evaluating these types of structures would gain attention by many researchers.

In the current study, with the assistance of Nonlinear Time History Analysis (NTHA) and Fourier Analysis (FA), the seismic assessments for evaluating the performance of a building in the loading period of the structure and the selected earthquake records by combining the responses of them and utilizing the Damage Content (DC) parameter to view the circumstance of the structure in every step of the seismic loading have been introduced. The introduced method cooperating with Incremental Dynamic Analysis (IDA) may provide more precise information about the lateral behavior of any other kinds of structures at every stage of surveying. Historical monuments located in seismically active places are the most vulnerable heritages of the commonwealths (Cavalagli et al., 2017; Pellegrini et al., 2018; Severini et al., 2018; Öztürk et al., 2019; Zampieri et al., 2019). Since the Middle East is the host for considerable numbers of historical monuments, especially masonry space domes, this study is involved with the seismic assessment of the Timche Haj-Mohammad-Qoli of Tabriz Historic Bazaar, which is considered a world heritage by UNESCO.

3 Masonry Space Structures

Jamshid Al-Kashani, the famous mathematician and astronomer in the early 15th Century to design the Ulug Beg observatory with larger spans, provided a specific definition of arches and vaults, and gather related knowledge in a book named Meftah Al-Hesab (Al-Kashani, 1427). From his viewpoint, the arch is a curved shape that has been used for the spanning of the gap and has a considerable depth. He redefined Chefd1 and Azj2 architectural terminologies. The final chapter of the study was devoted to Chefd and its definitions. There is also valuable information regarding the drawing arches and length of spans they can bear. Graham Bell introduced the tetrahedral space frame as an appropriate form for three-dimensional

---

1 - Arch in Middle Eastern term
2 - Vault in Middle Eastern term (derived from Persian word means curved branch)
structures, which was a significant breakthrough for space structures (Bell, 1903). After the invention of new types of space structures, the definition of space structure improved. IASS Committee defined the space frame as a structural system, assembled of linear elements so arranged that the loads are transferred in a three-dimensional manner. The constituent elements may be two-dimensional in some cases. Macroscopically, a space frame often takes the form of a flat or curved surface (IASS, 1984). The formal definition for space structures has been given by Nooshin (1998), in which the term space structure refers to a structural system that involves three dimensions. According to these definitions, equal presence in three dimensions is the most outstanding feature of the space structures. This definition resides considerable extent of the historical structures in the circle of space structures. Masonry domes such as Karbandi should consider a space structure as an instance of this definition. Before introducing steel as a load-bearing element in engineering, masonries were the dominant materials for structural uses, especially in ancient civilizations like Iran, which contains numerous masterworks like bazaar complexes, mosques, bridges, etc., the monuments have erected by masonries.

3.1 Karbandi

Based on the precision, Al-Kashani defined 5 types of Chefds, the first of which is a circle divided into six parts that can cover spans with a 5.2 m length. The second type also has a circular shape divided into eight equal sections. The angle between these sections is 45°. The Chefd can bear the loads of spans with a length of 5.2 m to 15.6 m. The third type is obtained by dividing the span radius into 8 parts that lead to getting the center points of the bottom and top arches (Memarian et al., 2014). This type of arch is suitable to use in spans of more than 10.4 m lengths. The definition of Chefd type four and five would also be traced in Meftah Al-Hesab (Al-Kashani, 1427), correspondingly. The climax of composition in aesthetic and structural applicability could be reflected in the Chefd type five, in which to get the vault, two lines are directed to the span Pakar¹.

---

¹ - Impost
Figure 1: Redefined and introduced 5 widespread Chefd types according to the mathematical calculations of Al-Kashani. All of the ancient arch types were reordered and mathematically reconsidered, for the possibility of utilization, in the upcoming future of that era. The obtained results were assembled in the book named Key to Arithmetic.

The described Chefds in Meftah Al-Hesab and their use in harmony become the key basis information of Karbandi in next generations. In Figure 1 the Chefd types categorized by Al-Kashani (1427) demonstrated.

According to Papadopoulo and Jazanî (1989), Karbandi derived from rolling squares inscribed in a circle. Sharbaf (2006), along with Pirnia and Bozorgmehri (2006), categorized Karbandi into general Shaghooli\(^1\) and Sarseft\(^2\) types with the shape of hexadecagon and other geometrical shapes in a simple form or made from the intersection of two shapes. Lu and Steinhardt (2007) introduced Karbandi as an advanced architectural approach of transferring loads resulted from the premier knowledge of geometry and mathematics of its founders. Theoretical geometry mainly focuses on lines surfaces area of three-dimensional objects, and the essence of practical geometry is to implement the science of geometry in wood, metal, brick, and other tangible things (Papadopoulo and Jazanî, 1989). Garofalo (2016) introduced Karbandi as a kind of arched shape roofing based on Islamic star patterns. In brief, Karbandi is employing

---

1 - Plummet in Middle Eastern term (derived from Azerbaijani word)
2 - Out of plumb in Middle Eastern term (derived from Persian word)
the geometrical pattern of one or multiple Chefs in harmony and aspect of subtlety, the outcomes of which would have a structural performance and application to the users. Examples of Karbandis consist of the Mausoleum of Baba Tahir (Hamadan, Iran), Mir-i-Arab Madrasa (Bukhara, Uzbekistan), Haidarzadeh Museum of Coin and Anthropology (Tabriz, Azerbaijan), etc. Karbandis widely used in the entrances, arcades, and intersections of historical Bazaars. Tabriz historical bazaar is one of the most landmark bazaars with numerous Karbandis in its different parts. The continuity of using Karbandi is highly demanded in contemporary Islamic Architecture but requires a precise understanding of geometrical and structural aspects of ancient Karbandis to achieve a modern design and construction strategy to meet new requirements.

3.2 Timche Haj-Mohammad-Qoli

The building of Timche Haj-Mohammad-Qoli dates back to the early Qajar dynasty (the 1810s). The architectural evaluation of the type and design of the vaults aesthetically concurred with that of the infrastructure fundamentals of that period. Although the design principles and rationale may root back to the basis of Karbandi, astronomical requirements, space creation, and the philosophy of using Chefd as an accurate object of arch creation are also plays a crucial role, the further details of which have been formerly discussed.

3.2.1 Location

Tabriz Historic Bazaar Complex consists of numerous masonry space structures and various fundamental features of masonry structural systems. These features include architectural and structural morphologies, force flow and force management in the masonries, and perception of the collaboration among structural elements. Timche\(^1\) Haj-Mohammad-Qoli, with 11.5 meters diameter, is one of the covered spaces masterpieces of Tabriz bazaar with a sophisticated configuration of spatial masonry intersecting arches and selected for further evaluations in the current study. The Bazaar has various Timches and Rastehs\(^2\), which have covered with different masonry curved surfaces such as dome, vault, and Karbandi. The

---

1 - Covered trade places
2 - Covered trade passages
applications of Timches in the past were for the products wholesale. The coverture of all the Timches is an
arched shape foremost has been made by using Karbandi. In Figure 2, the location and section cuts of the
sophisticated Karbandi of the Timche Haj-Mohammad-Qoli presented. It has been located in the southern
part of the Historical Bazar. The main part of the Timche is located at the northeast corner of the Bazar
Complex. It has accessibilities from four sides. Appropriate connectivity with neighbor elements,
especially in the east-west axis, makes the strong bond with Bazar texture. This Timche has been built
based on orderly and comprehensive geometry and symmetrical shape on the north-south and west-east
axis.
Figure 2: Timche Haj-Mohammad-Goli of Tabriz Historical Bazaar (a) Geographical location of the site; (b) 2D plan view and cross sectional views
3.2.2 Geometry

The skeleton of Timche had been constructed by using Karbandi. The blueprint of the structure is the combination of an eight\textsuperscript{1} and a half-eight\textsuperscript{2} plan that is rotated around the central point with the 45° angle and coincided with the pattern at the beginning. Harmoniously, the roofing system consisted of a dome shape arch at the center and four half–arches on the sides. The middle part was filled with sixteen-wing Shamseh\textsuperscript{3} and brick domes with simple ornamentations. In comparison to other Karbandis in Bazar Complex, the Timche Haj-Mohammad-Qoli has the most sophisticated architectural structure. Regarding Sharbaf (2006), the Timche could consider as Sarseft Karbandi made from the intersection of two simple hexadecagons, though its geometry is slightly different. The geometry creation leads to the development of 16 intersecting equal chords between the dividing points with identical arches located on each chord. Parallel to the skeleton forms the interior face of the Timche, another Chefd network has been used in the exterior skeleton of the structure unseen from inside. The employed arches in the structural elements of this Karbandi, according to the categorization of Al-Kashani (1427), most probably reside in 2nd or 3rd Chefd type. Figure 3 presented the final feature of the Karbandi of Timche Haj-Mohammad-Qoli from inner and outer perspectives.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure3.png}
\caption{The structural and architectural appearance of the Timche Haj-Mohammad-Qoli’s Karbandi; (a) The hidden ribbed arches outside of the building increasing the load-bearing capacity of the Karbandi; (b) A final feature of Karbandi in which two}
\end{figure}

\begin{flushright}
1 - Islamic architectural plan type  
2 - Islamic architectural plan type  
3 - The crown part of the Karbandi where the dome-like part of the roofing begins (derived from an Arabic word that means sun-like)
\end{flushright}
The structural evaluations of any system may consist of both non-destructive and destructive tests and there would be difficulties in performing destructive tests according to the historical value of the Timche. Thus, up to the greatest extent, the assessments conducted to the non-destructive tests. The geometrical faces, spans length, arches dimensions, dome height, and the other requirements regarding the architectural principles were measured. The visual inspections determined the material properties used within the masonry consisted of clay brick and lime mortar. While getting direct samples from the masonry was ineligible, a request for acquiring unit and joint samples was officially submitted to the Cultural Heritage, Handicrafts, and Tourism Organization of East Azerbaijan. The bureau yielded limited numbers of masonry units extracted from the Timche and the lime mortar combination used inside the building, which is not presented due to passive defense factors. To obtain the characteristics of the materials, masonry prisms were constructed in the laboratory according to the instructions of the American Society for Testing and Materials (ASTM).

### 4.1 Material Properties

Determining the tensile behavior of masonry prisms regarding the principles of both ASTM E518/E518M-10 (2010) and ASTM E519/E519M-10 (2010) standards are viable. Although, since ASTM E519/E519M-10 (2010) is not applicable for lime mortars, the tensile and bending characteristics of the studied materials regarding the principles of ASTM E518/E518M-10 (2010) have determined. The compressive behavior of the studied materials has performed according to ASTM C1314-07 (2007). In Table 1 and Table 2, details of the experimental outcomes of the studied samples have presented for flexural bond strength and compressive strength, respectively. In Figure 4, laboratory tests for determining the compressive strength and flexural bond strength of two specimens have been brought.
Table 1: Laboratory performed tests to determine the flexural bond strength of the studied samples according to ASTM E518/E518M-10 (2010)

| Specimen No. | Test Method | Age at Test (Days) | Avg. Width (mm) | Avg. Depth (mm) | Avg. Length (mm) | Weight (kN) | Max Load (kN) | Strength (MPa) |
|--------------|-------------|--------------------|-----------------|-----------------|------------------|-------------|---------------|---------------|
| 1            | A *         | 28                 | 201             | 101             | 460              | 0.152       | 0.260         | 0.084         |
| 2            | A *         | 28                 | 200             | 100             | 460              | 0.150       | 0.203         | 0.073         |
| 3            | A *         | 28                 | 202             | 101             | 461              | 0.155       | 0.257         | 0.083         |
| 4            | A *         | 28                 | 201             | 102             | 460              | 0.153       | 0.277         | 0.086         |
| 5            | A *         | 28                 | 201             | 99              | 460              | 0.151       | 0.188         | 0.070         |

Average - - 201 101 460 0.152 0.237 0.079
Deviation - - 0.71 1.14 0.45 0.002 0.039 0.007
CV ** - - 0.004 0.011 0.001 0.013 0.165 0.092

* It is a third-point loading method that was discussed in ASTM E518 with further details.

** Coefficient of Variation.

Table 2: Laboratory performed tests to determine the compressive strength of the studied samples according to ASTM C1314-07 (2007)

| Specimen No. | Age at Test (Days) | Avg. Width (mm) | Avg. Height (mm) | Avg. Length (mm) | h_p/t_p Correction Factor * | Max Load (kN) | Strength (MPa) |
|--------------|--------------------|-----------------|-----------------|------------------|-----------------------------|---------------|---------------|
| 1            | 28                 | 101             | 315             | 200              | 1.077                       | 47.4          | 2.53          |
| 2            | 28                 | 102             | 316             | 201              | 1.076                       | 51.8          | 2.72          |
| 3            | 28                 | 102             | 318             | 200              | 1.077                       | 48.1          | 2.54          |
| 4            | 28                 | 101             | 318             | 199              | 1.079                       | 51.0          | 2.74          |
| 5            | 28                 | 100             | 318             | 200              | 1.084                       | 50.1          | 2.72          |

Average - 101 317 200 1.079 50 2.65
Deviation - 0.84 1.41 0.71 0.003 1.88 0.10
CV ** - 0.008 0.004 0.004 0.003 0.038 0.040

* It is the ratio of the prism height to the least lateral dimension of it according to ASTM C1314.

** Coefficient of Variation.

Due to the erosion, weathering, corrosion, etc., factors, and the historical content of the perusing structure, the accuracy of the excerpted outcomes from experimentally evaluated specimens may be affected. Albeit,
since the precision of any other non-destructive tests including but not limited to Schmidt hammer test, ultraviolet test, ultrasonic test, or the imposed damage from destructive tests like coring could not be acceptable, the prepared and tested samples regarding the principles of ASTM E518/E518M-10 (2010), were still advantageous. Therefore, they selected the criteria for evaluating the results. The experimental results and research outcomes of the compressive strength for masonry are in good agreement. The same condition also observed in the ultimate strain obtained from indirect tensile tests and extracted values from the studies. By taking this into account, the obtained values from the lab utilized for numerical modeling of the material behavior.

![Material tests of prepared samples](image)

Figure 4: Material tests of prepared samples; (a) Compression strength test according to ASTM C1314-11 (2011); (b) Flexural bond strength test according to ASTM E518/E518M-10 (2010).

The mechanical properties of the materials utilized within the masonry structure of the Karbandi, referring to the outcomes provided in Table 1 and Table 2 demonstrates diminutive deviations. Therefore, there is a feasibility to adopt failure criterions in numerical simulation. Thus, Concrete Damage Plasticity (CDP) criterion for the masonry material was implemented in numerical analysis. The stress-strain curve for the employed material has been illustrated in Figure 5.
4.2 Record Selection Criteria

Regarding the principles of BHRC (2015), the strong ground motions were chosen by considering their magnitude, fault type, epicentral distance, hypocentral distance, soil type, and effective duration. Since a notable number of historical strong ground motions that took place in Tabriz are estimated to range between 5 to 8 Richter in magnitude, all the selected records have a magnitude larger than 6 Richter. The most active fault ilk in the Timche surrounding and geographical map is strike-slip, which is located in the northern part of the city on the seismic belt of Alp-Himalaya. Most of the recorded earthquakes in the studied zone have a hypocentral depth of 10 to 20 kilometers, and the perpendicular distance of the site from the fault is about 15 kilometers. The shear wave velocity of the soil in the site ranged between 175m/s to 375m/s. Thus, the records with a hypocentral distance of 0 to 50 kilometers and epicentral distance of 0 to 30 kilometers with aforementioned analogous shear velocities are the focal point of the search for similar records. The records considered in the probing procedure should have at least 10 seconds of effective duration, referring to the principles of BHRC (2015). Since it is located in the vicinity of both major and minor faults, almost every constructed structure inside the city was prone to both near-field and far-field earthquakes. This study also considered near-field records for evaluating the effects. The epicentral distance and high unorthodox values in the velocity time history were considered the signs of only-pulsed records in various research studies (Baker, 2008; Panella et al., 2017; Kohrangi et al., 2019). In this study, the most prevailing selection method has been used, in which the epicentral distances lesser than 10 kilometers were considered only-pulsed records. Eventually, 20 records of strong ground
motions, half of which are only-pulsed, were selected for the current study. Since all the fallen severe earthquakes in Tabriz were belonged to the era when no measuring devices and no information about the site records exists, the relevant data was collected from the Pacific Earthquake Engineering (PEER) database. It is essential to mention 16 of the selected records are conjugated, which means the record data is extracted from two different stations to resemble the near-field and the far-field effects of the selected records. The characteristics of selected records for both near-field and far-field ground motions have shown in Table 3.
| Record Name                      | Year | Station                          | Abb  | PGA (g) | Duration (s) |
|----------------------------------|------|----------------------------------|------|---------|--------------|
| **Far field Strong Ground Motions** |      |                                  |      |         |              |
| "Coyote Lake"                    | 1979 | "San Juan Bautista_24 Polk St"  | CSN  | 0.118   | 28.4         |
| "Darfield_ New Zealand"          | 2010 | "Christchurch Cashmere High School" | DCN  | 0.297   | 100.8        |
| "El Mayor-Cucapah_Mexico"        | 2010 | "El Centro - Meloland Geot. Array" | EEN  | 0.439   | 87.4         |
| "Imperial Valley-06"             | 1979 | "Calexico Fire Station"          | ICN  | 0.277   | 37.8         |
| "Kobe_ Japan"                    | 1995 | "Kakogawa"                       | KKN  | 0.324   | 40.9         |
| "Landers"                        | 1992 | "Coolwater"                      | LCN  | 0.417   | 27.9         |
| "Superstition Hills-02"          | 1987 | "Brawley Airport"                | SBN  | 0.284   | 27.9         |
| "Westmorland"                    | 1981 | "Niland Fire Station"            | WNN  | 0.176   | 39.9         |
| "Parkfield"                      | 1966 | "Cholame - Shandon Array #8"     | PCN  | 0.272   | 25.1         |
| "Duzce_ Turkey"                  | 1999 | "Lamont 1062"                    | DLN  | 0.259   | 42.2         |
| **Near field Strong Ground Motions** |      |                                  |      |         |              |
| "Coyote Lake"                    | 1979 | "Gilroy Array #4"                | CGO  | 0.422   | 27.1         |
| "Darfield_ New Zealand"          | 2010 | "Christchurch Botanical Gardens" | DCO  | 0.190   | 63.8         |
| "El Mayor-Cucapah_Mexico"        | 2010 | "Westside Elementary School"     | EWO  | 0.281   | 103.2        |
| "Imperial Valley-06"             | 1979 | "Agrarias"                       | IAO  | 0.472   | 28.4         |
| "Kobe_ Japan"                    | 1995 | "Port Island (0 m)"              | KPO  | 0.567   | 41.9         |
| "Landers"                        | 1992 | "Yermo Fire Station"             | LYO  | 0.245   | 43.9         |
| "Superstition Hills-02"          | 1987 | "Kornbloom Road (temp)"          | SKO  | 0.432   | 22.3         |
| "Westmorland"                    | 1981 | "Parachute Test Site"            | WPO  | 0.232   | 41.6         |
| "Parkfield-02_CAO"               | 2004 | "Parkfield - Fault Zone 9"       | PPO  | 0.153   | 21.1         |
| "Kocaeli_ Turkey"                | 1999 | "Yarimca"                        | KYO  | 0.322   | 34.9         |

Each record was composed of vertical and two perpendicular pivots. Standards and design codes, including but not limited to (Eurocode 8, 2005; ASCE/SEI 7-10, 2013; Standard 2800-15, 2015) normalize earthquake records for nonlinear dynamic analysis by scaling PGA of the ground motion to 1.0.
g and applying the modification factor to them that obtained from comparing the convoluted response spectrum of the selected records to the standard design spectrum in the determined range of Time period for the studied structures. In this study, the records according to the principles of BHRC (2015) have been scaled and implemented in numerical modeling using ABAQUS FEM software. After the record normalization, the stress and strain outputs of the Karbandi subjected to the near and far-field records have been extracted and brought for further consideration, as explained in the Time History Analysis section.

4.3 The Failure Criterion

The failure mechanism in skeleton structures, took place when the formation of plastic hinges makes the whole structure or some parts of it unstable, referring to the principles of structural mechanics. Since masonry structures are continuous nonhomogeneous elements with low integrity, the determination of failure mechanism for them is more complicated than that of one-dimensional elements. According to the experimental and numerical outcomes of many pieces of research (Andreas, 1996; Noor-E-Khuda et al., 2016; Bui et al., 2019), the ultimate failure of the masonry elements took place in a plane or planes of crack formation where some part of the masonry become unstable. Hence, it can assume that the failure of the Timche occurs following the formation of the first plane of failure, which leads to formation of a mechanism in some or all sections of it. The failure of masonry elements is the ultimate plastic strain of masonry prisms in compression and flexure extracted from experimental results. According to CDP criterion, the plastic strain respected to masonry failure in compression and tension was considered to be 0.00389 and 0.00032, respectively. These amounts were used for resembling the compressive and tensile behavior of masonry elements in numerical simulations. The corresponding failure drift in Nonlinear static analysis was considered to be the failure drift of the structure, in which it was comprehensively collapsed, and the analogous Damage Content (DC) of it was deemed equal to one. The zero point of the Damage Content (DC) is initiated when the plastic behavior of the structure corresponds to the yield point of the bilinear pushover curve of the Timche. The failure content of the Timche shall subsequently evaluate for the near-field and the far-field earthquakes.
5  Numerical Analysis

Due to structural complexity and controversial time costs of micro-modeling, numerical modeling of the masonry dome could reside in borderlines of macro-modeling strategy. In most cases because of the far beyond complexity and/or time-consuming procedure of the other methods, the macro-modeling approach was considered more efficacious since it can distinguish the response of large-scale structural elements and building systems with ease (Lourenço et al., 1995; Baloevic et al., 2016; Pantò et al., 2016). With regard to its acceptable accuracy, the CDP criterion was used for simulating the post-failure behavior of the masonry, the related parameters of which adjusted referring to Section 4.1(Material Properties). Static General analysis to determine the effects of the gravity loads (linear static analysis), Nonlinear Implicit analysis for the nonlinear pushover, and Nonlinear Explicit analysis for the nonlinear time history evaluations were employed through the related modules of ABAQUS FEM software. According to provided principles in Section 4.3 (Failure Criterion) and mechanical concepts, nonlinear pushover analysis was employed to get the bilinear curve and utilize in nonlinear dynamic analysis. The results obtained from the aforementioned curves, more specifically the displacement values related to the equivalent yielding and ultimate load-bearing points, were used in every stage of the dynamic appraisal deformation curves and obtaining the final DC value of the studied earthquake records. Due to the complexity of the masonry, the double-precision analyzing approach was employed to perform the explicit analysis. All components of the model were discretized using 4-node 3D linear tetrahedron elements (C3D4). The numerical model consisted of 23961 meshes. The graphical scheme of the FEM model has shown in Figure 6 Places where arch pillars posed on the earth considered as fixed supported boundary conditions.
The masonry material used for dome, according to experimentally assessed masonry units and mortar samples (Section 4.1), was assumed to have an Elasticity modulus of 2690MPa, and Poisson's ratio of 0.15. For further details of calculating masonry parameters, Eurocode 6 (2005), Kaushik et al. (2007), and Kmiecik and Kamiński (2011) are to be referred. The models were subjected to 8000 N/m$^2$ vertical loads, which derived from the structure's self-weight (the dome thickness is 0.15 m), and the roof loads consist of precipitation and human loads.

### 5.1 Linear Static Analysis

The dome behavior, natural period, and its reaction to the vertical loads were investigated by using linear static analysis. The linear static analysis was carried out by considering the self-weight of the overall structure and rain and snow loads. Analysis results were surveyed in terms of maximum stress variation and the stress flow of the dome. Stress distribution due to gravity loads in the masonry space dome demonstrates that the maximum values of compressive and tensile stresses in both surfaces of the elements were taking place at the eight main load-bearing pillars of the Timche, which has displayed in Figure 7. The stress flow in Timche to a better understanding of the structure mechanism have presented in Figure 8. According to the figure, it can be deduced the force flow transmits from minor arches to the main vaults and from them to the load-bearing pillars. Results also depict while the compressive stresses are dominant in the majority of members, tension stresses are negligible. Therefore, dominated stress distribution among the members indicates, despite the level of scientific knowledge during the building era, the structure was constructed skillfully.
Figure 7: Distribution of compressive and tensile stresses by employing Tresca criterion and maximum principal stress in gravity direction.

Figure 8: Stress flow within the main load-bearing elements of the Timche by considering 3D and gravity directions. The stress flow concentration in the corners of the Timche to convey the final flow to the endpoints in the gravity direction is considerable.

The structure period was obtained by Modal Analysis. The frequency range to perform the analysis by assuming the first natural period of the structure was assigned between 0.5 Hz and 20 Hz. The first frequency of the Timche was around 6.15 Hz that respects 0.163 Sec to the primary period of the
structure. The analysis results due to gravity loads employed in Nonlinear Pushover Analysis. The graphic scheme of the first 10 mode shapes of the Karbandi has been presented in Figure 9. In Table 4, all the information related to the initial 10 mode shapes of the Karbandi has been brought.

Table 4 The modal analysis results for the Timche Haj-Mohammad-Qoli including Circular Frequencies and Mode Types

| Mode No. | Frequency (Hz) | Period (s) | Circular Frequency (Rad/s) | Mode Type     |
|----------|----------------|------------|-----------------------------|---------------|
| 1        | 6.151          | 0.163      | 38.648                      | Lateral-Dir 1 |
| 2        | 6.1516         | 0.163      | 38.652                      | Lateral-Dir 2 |
| 3        | 7.0919         | 0.141      | 44.560                      | Vertical      |
| 4        | 7.2265         | 0.138      | 45.405                      | Vertical      |
| 5        | 7.2581         | 0.138      | 45.604                      | Flexural      |
| 6        | 11.173         | 0.090      | 70.202                      | Lateral-Dir 1 |
| 7        | 11.173         | 0.090      | 70.202                      | Lateral-Dir 2 |
| 8        | 12.36          | 0.081      | 77.660                      | Torsional     |
| 9        | 13.318         | 0.075      | 83.679                      | Vertical      |
| 10       | 13.32          | 0.075      | 83.692                      | Vertical      |

Due to symmetry in the shape of the Karbandi, the concatenating lateral mode shapes in frequency analysis were identical. The same observations have been sighted and traced in higher modes.
Figure 9: The first 10 mode shapes of the Karbandi of Timche Haj-Mohammad-Qoli
5.2 Nonlinear Pushover Analysis

A linearized pushover curve could render an approximation to the yield and collapse point of the Karbandi. To this purpose, considering the plastic strain of the masonry, up to the point the first plane of failure was came into view the displacement-controlled lateral loads applied to the structure. In this regard, the ultimate strain extracted from the CDP criterion that considered the final failure point of the masonry could also be considered the point of failure in the formation of the failure plane. The bilinear curve shows the capacity of the Karbandi under the lateral load and corresponding displacement subjected to that. Since the probability of the failure in both directions was assumable, the pushover curve and related bilinear curve in both directions were extracted. Resultantly the curve with a lesser maximum failure load was considered the ultimate failure curve of the structure. In Figure 10 linearized pushover curve of the Timche in two perpendicular directions has been illustrated. In Figure 11, the formation of the failure plane concerning the attributed plastic strain in nonlinear pushover analysis for the idealized directions has presented.

![Bilinear Curve (Direction 1)](image1)

![Bilinear Curve (Direction 2)](image2)

Figure 10: Bilinear Curve of the studied Dome in the perpendicular Directions

Due to lesser load-bearing parameters in presented results, direction 1 should be considered as the critical direction. In Table 5, by assuming the yield and failure points the simplified bilinear curves and corresponding load-bearing strength of the structure has brought.
Table 5: Bilinear curve and corresponding strength of the nonlinear pushover analysis in perpendicular directions

| Bilinear Curve in Direction 1 | Bilinear Curve in Direction 2 |
|-----------------------------|-----------------------------|
| Displacement (mm) | Load (kN) | Displacement (mm) | Load (kN) |
| 0 | 0 | 0 | 0 |
| 13 | 668.7 | 13 | 868.4 |
| 73 | 586.0 | 61 | 748.1 |

5.3 Time History Analysis

The mixed average and envelope response spectrum curves presented in Figure 12 compare the standard design spectrum of BHRC (2015), referring to the selected records. According to the scaling principles proffered by BHRC (2015), the mixed response spectrum of the records between 0.2T and 1.5T, in which
T is the natural period of the structure, may not be less than the 90% of the standard spectrum or modification coefficient. Since the scaling procedure for determining the modification coefficient are almost the same, further principles can also be traced from ASCE/SEI 7-10 (2013) and Eurocode 8 (2005). Through the numerical time history analysis, the rest of required data for extraction of the DC value of studied records were obtained. The plastic strain at the final stage of loading for the studied records has been illustrated in Figure 13.

Figure 12: The mixed average and envelope response spectrum of the near-field and far-field records compared with the standard design spectrum of BHRC (2015)
The records with corresponding critical displacement in the crown of the Timche, referring to their significance and controversy for unexpected deformation have chosen for more assiduous assessment. From the selected records three belong to the only-pulsed category (DCO, EWO, and KYO) and one to the non-pulse records (DCN), which is the most critical record in that class. The displacement of the control point and related Energy curve of the studied structure for three near field and one far field earthquake has been presented in Figure 14.
Figure 14: The recorded displacement of the control point (crown of the Karbandi) (Left Column) and corresponding Dissipation (Strain and Kinetic) Energy (Right Column) for DCO, EWO, and KYO (near-field records) and DCN (far-field record)

6 Damage Content

As previously explained, the damaged content was computed according to provided principles in Section 5 (Numerical Analysis). Resultantly the DC at each stage of earthquake application with considering the
corresponding displacement, the final failure of which is assuming according to the formation of analogous plastic strain failure plane, obtained and for perpendicular directions in both far-field and near-field earthquakes respectively presented in Figure 15 and Figure 16. The failure in these assessments was assigned when the DC reaches 1.
Figure 15: DC value of far-field ground motions in perpendicular directions.
Figure 16: DC value of near-field ground motions in perpendicular directions
Prior to the eventuation of the strong ground motion, failure in the perpendicular directions of the Timche axis was taken into place for the DCN in the far-field plus the DCO, EWO, KPO, KYO, and WPO in near-field earthquake cases. Failures in the X axis (1\textsuperscript{st} direction) in most of the cases precede the Y axis (2\textsuperscript{nd} direction) and exist in all the cases that happened in both axes. In Figure 17, the crown drift of the Karbandi for X, Y, and vertical directions subjected to the near-field and far-field ground motions have been depicted.

DCN among the studied far-field records has the critical drift in all the directions, and its massive share causes that all displacements to drop below the mean average of the calculated displacements. On the other hand, KYO records in all directions along with DCO in the Y direction and EWO in the vertical
direction have had critical drift values. In Figure 18, the strain and kinetic energy differences for the studied records have been shown.

Maximum kinetic energy and minimum strain energy in far-field earthquakes belong to the SBN, while the same extremums in near-field strong motions respectively belong to the SKO and PPO records. Due to the difference in the records’ duration, to obtain the average, minimum, and maximum values in a comparable condition, Fast Fourier Transform (FFT) has employed, referring to which the time domain was converted to the frequency domain. The method makes it possible to compare the damage contents of the records. Half of the frequency-domain curves were provided, referring to the symmetry in FFT outcomes and for the convenience in better assessment of the outcomes. Accordingly, relevant curves for both near-field and far-field records have been illustrated in Figure 19.
Figure 19: Fast Fourier Transfer (FFT) of Damage Content (DC) of the near-field and the far-field strong ground motions

The median, mean, minimum, and maximum of the transformed contents have been brought back to the time domain using inverse FFT afterward. Employing IFFT leads to the ultimate mixed damage content value, in which the X-axis and Y-axis are the normalized time and the damaged content, respectively.
Figure 20: Inverse Fast Fourier Transfer (IFFT) of Damage Content (DC) of the near-field and the far-field strong ground motions

Figure 20 provides DC values for Timche subjected to near-field and far-field ground motions. The values related to the maximum, median, mean, and minimum values of the DC have been extracted. Despite the similarity in occurrence time for the maximum collapse curve of the structure, there are considerable differences in near-field and far-field DC values. For the far-field DC values, there is a gap between the maximum and median curves that indicates even though the structure collapsed in some records, it retained its functionality in the others. In opposite, for the DC value in the near-field earthquakes, the gap between maximum, median, and mean are very diminutive, and most of the records impose heavy damages to the structure. Minimum values in the near-field IFFT curve are more than fifty percent greater than that of the far-field IFFT curves. Considering all the above-mentioned facts, the results emphasize the importance of the near-field records due to the incurred damage. According to the outcomes, the studied structure for a hundred-second lasting far-field earthquake in 50% of the cases may survive with severe damages; however, for the same near-field earthquake cannot survive for more than 40 seconds. In other words, if a near-field earthquake for the studied Timche has applied most probably before 0.4 of the earthquake occurrence time, the failure and collapse of the structure may take place.

7 Conclusion

The study relates to employ an alternative approach in evaluating historical structural systems using FFT, nonlinear static analysis, and nonlinear dynamic analysis. Accordingly, the structural system of Timche Haj-Mohammad-Qoli concerning the architectural concepts, executing aspects (with probing the historical
documents and resources), and current structural stance have been evaluated. The damage content (DC) concept regarding the mechanical characteristics of the used material, including plastic strain, failure plane, and ultimate load-bearing capacity, along with drift value, record selection criteria, and architectural aspects, has been employed. The present system undertakes the vertically applied gravity loads and transfers the force flow with convenience. However, the performed pushover analysis demonstrates the vulnerability of the structure to the lateral loads due to the limited drift content of the structural system. At last, a unified structural response in the frequency domain by merging the inverted damage content curves have obtained since the damage content of the structure in various records may alter, and differences in earthquake durations hinder the possibility of combining the structural response in the time domain. The residual outcome reversed to the time domain to make them structurally discussable. The most landmarks results of the study, according to the aforementioned abbreviation, are as follows:

- The idea of Karbandi from architectural feature, at least could be traced back to the early 15th century, when eminent mathematician Ghiyath Al-Din Jamshid Masud Al-Kashani, to establish the fundamentals of arches with longer spans redefined and categorized the ancient science of the vaults and arches construction. The newly acquired science was used to build the observatory of Ulug Beg, which afterward became one of the extraordinary astronomical science discovery places of that era. The byproduct of this breakthrough led to employing geometrical patterns to utilize in places that requires long spans and broader spaces like mosques and covered bazaars, more specifically in cold habitats. The esthetical reflection of this achievement inspired the widespread use of Karbandi for ornamental purposes.

- The historical structure of the Timche was designed in a way to retain all the possible privileges of the compressive strength of masonry units in a manner that the least tensile strength due to the gravity loads impose to the structure. The system has a proper reaction to the lateral loads in the incipient stages of lateral deformation. However, imposing further lateral deformation exposes the masonry to the failure.

- An alternative method of evaluating structural systems, especially for those with higher sensitivity in their façade or other featural aspects due to their historical importance, aesthetical attraction, etc., with applying Fourier Transform to the damage ratio of time history outcome in the frequency domain has been
introduced. The behavior of the vulnerable structures due to earthquake excitations by employing the introduced method along with IDA analysis may most properly determine the failure points of the structural system; hence, the required augmentation, including retrofitting, rehabilitation, and strengthening, with the least interception to the mechanical characteristics of the system, could be employed.

As expected, the obtained DC from the far-field earthquakes has a lesser ratio comparing to the near-field earthquakes. It is assumable due to their velocity contents, which most probably affect the structures with short period, the near-field strong ground motions for the low-rise structures have more significant repercussions. According to the results, 3 out of 10 far-field records cause failure in the structure, including one that took place in the perpendicular directions, whereas 7 out of 10 near-field earthquakes lead to the collapse-correspondent failure in the Timche, with five occurred in perpendicular directions. Regarding the outcomes, a near-field earthquake may finally lead to the structural collapse during the initial stages (not more than 40% of the earthquake duration), whilst after the eventuation of a far-field earthquake, there is a 20% hope that the structure survives with a close to collapse and severely damaged remnants.

The reliability of the median curve for unpredictable events such as earthquakes shall be more than that of the mean curve since the inverse median curve statistically represents more than 50% of the studied records, whilst the mean curve mathematically is the average value of the DC for the same records and cannot provide that assurance. On average the DC value for the median curve in the far-field and near-field earthquakes is around 15.7% and 14.9% more than its peer amounts in the mean curves, respectively.

8 Data Availability

Since some outcomes of this study didn't publish in any other Journal, all data, models, and the results that support the findings of this study are available from the corresponding author upon reasonable request.
References

- 36 CFR 68, the Secretary of the Interior's Standards for the Treatment of Historic Properties (2012), Code of Federal Regulations, 16 U.S.C. 470et seq.

- Al-Kashani, G. J. (1427), Meftah Al-Hesab (The Key to Arithmetic), Samarkand, Timurid Empire (In Arabic).

- ACI (2004). ACI 228.2R-98: Nondestructive Test Methods for Evaluation of Concrete in Structures, American Concrete Institute (ACI), Farmington Hills, MI.

- ACI (2007). ACI 364.1R-07: Guide for Evaluation of Concrete Structures before Rehabilitation, American Concrete Institute (ACI), Farmington Hills, MI.

- ACI (2008). ACI 201.1R-08: Guide for Conducting a Visual Inspection of Concrete in Service, American Concrete Institute (ACI), Farmington Hills, MI.

- ACI (2011). ACI 369R-11: Guide for Seismic Rehabilitation of Existing Concrete Frame Buildings and Commentary, American Concrete Institute (ACI), Farmington Hills, MI.

- ACI (2011), ACI 530: Building Code Requirements and Specification for Masonry Structures, Masonry Standards Joint Committee (MSJC), Reston, VA.

- ACI (2013). ACI 562M-13: Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings and Commentary, American Concrete Institute (ACI), Farmington Hills, MI.

- Aghabeigi, P., Mahmoudi, R., Ahani, E., & Hosseinian Ahangarnazhad, B. (2020). Seismic Assessment and Retrofitting of the Masonry Building of Mozaffarieh Timche in Tabriz Historic Bazaar. International Journal of Architectural Heritage, 1-26.

- Ahani, E., Mousavi, M. N., Rafezy, B., & Osmanzadeh, F. (2019). Effects of central opening in masonry infill on lateral behavior of intermediate RC frames. Advances in Civil Engineering Materials, 8(1), 23-42.
Ahmadi, M. P. (2014). A basic method for naming Persian Karbandis using a set of numbers. *Nexus network journal, 16*(2), 313-343.

Andreaus, U. (1996). Failure criteria for masonry panels under in-plane loading. *Journal of structural engineering, 122*(1), 37-46.

ASCE/Structural Engineering Institute. (2003). Seismic Evaluation of Existing Buildings (ASCE/SEI 31-03), American Society of Civil Engineers.

ASCE/Structural Engineering Institute. (2007). Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06), American Society of Civil Engineers.

ASCE/Structural Engineering Institute. (2013). Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10). American Society of Civil Engineers.

Asteris, P. G., Chronopoulos, M. P., Chrysostomou, C. Z., Varum, H., Plevris, V., Kyriakides, N., & Silva, V. (2014). Seismic vulnerability assessment of historical masonry structural systems. *Engineering Structures, 62*, 118-134.

Asteris, P. G., Moropoulou, A., Skentou, A. D., Apostolopoulou, M., Mohebkhah, A., Cavaleri, L., Rodrigues, H., & Varum, H. (2019). Stochastic vulnerability assessment of masonry structures: concepts, modeling and restoration aspects. *Applied Sciences, 9*(2), 243.

ASTM C39/C39M-21 (2021). Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, ASTM International, West Conshohocken, PA, [www.astm.org](http://www.astm.org), DOI: 10.1520/C0039_C0039M-21.

ASTM C42/C42M-20 (2020). Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete, ASTM International, West Conshohocken, PA, [www.astm.org](http://www.astm.org), DOI: 10.1520/C0042_C0042M-20.
- ASTM C496/C496M-17 (2017). Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, ASTM International, West Conshohocken, PA, [www.astm.org](http://www.astm.org), DOI: 10.1520/C0496_C0496M-17.

- ASTM C597-16 (2016). Standard Test Method for Pulse Velocity Through Concrete, ASTM International, West Conshohocken, PA, [www.astm.org](http://www.astm.org), DOI: 10.1520/C0597-16.

- ASTM C1314-07 (2007). Standard Test Method for Compressive Strength of Masonry Prisms, ASTM International, West Conshohocken, PA, [www.astm.org](http://www.astm.org), DOI: 10.1520/C1314-07.

- ASTM E518 / E518M-10, Standard Test Methods for Flexural Bond Strength of Masonry, ASTM International, West Conshohocken, PA, 2010, [www.astm.org](http://www.astm.org), DOI: 10.1520/E0518_E0518M-10.

- ASTM E519 / E519M-10, Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages, ASTM International, West Conshohocken, PA, 2010, [www.astm.org](http://www.astm.org), DOI: 10.1520/E0519_E0519M-10.

- Atamturktur, S., & Laman, J. A. (2012). Finite element model correlation and calibration of historic masonry monuments. *The Structural Design of Tall and Special Buildings*, 21(2), 96-113.

- ATC (1996). Seismic Evaluation and Retrofit of Concrete Building, Report (ATC-40), Applied Technology Council, California, Vol. 1, USA.

- Baloevic, G., Radnic, J., Matesan, D., Grgic, N., & Banovic, I. (2016). Comparison of developed numerical macro and micro masonry models for static and dynamic analysis of masonry-infilled steel frames. *Latin American Journal of Solids and Structures*, 13(12), 2251-2265.

- Baker, J. W. (2008). Identification of near-fault velocity pulses and prediction of resulting response spectra. *Geotechnical earthquake engineering and soil dynamics* IV (pp. 1-10).

- Bell, A. G. (1903). The tetrahedral principle in kite structure. *Judd & Detweiler*.

- Betti, M., & Galano, L. (2012). Seismic analysis of historic masonry buildings: the vicarious palace in Pescia (Italy). *Buildings*, 2(2), 63-82.
- BHRC (2015). Iranian code of practice for seismic resistance design of buildings, Standard 2800-15, 4th Edition, BHRC Publication No. S-253, Building & Housing Research Center, Tehran, Iran.

- Binda, L., & Saisi, A. (2005). Research on historic structures in seismic areas in Italy. *Progress in Structural Engineering and Materials*, 7(2), 71-85.

- Boscato, G., Pizzolato, M., Russo, S., & Tralli, A. (2014). Seismic behavior of a complex historical church in L’Aquila. *International Journal of Architectural Heritage*, 8(5), 718-757.

- Boughton, B., & Falconer, R. E. (2001). The strengthening and repair of masonry structures using the MARS system. In Proceedings of the International Conference on Structural Faults and Repair, London.

- Bozorgmehri, Z. (1992). Hendese Dar Me’mari (Geometry in Architecture). Tehran: Sazmane Mirase Farhangi-ye Keshvar (Iranian Cultural Heritage Organization).

- Brandonisio, G., Lucibello, G., Mele, E., & De Luca, A. (2013). Damage and performance evaluation of masonry churches in the 2009 L’Aquila earthquake. *Engineering Failure Analysis*, 34, 693-714.

- Bui, T. T., Limam, A., & Sarhosis, V. (2019). Failure analysis of masonry wall panels subjected to in-plane and out-of-plane loading using the discrete element method. *European Journal of Environmental and Civil Engineering*, 1-17.

- Carocci, C. F. (2012). Small centres damaged by 2009 L’Aquila earthquake: on site analyses of historical masonry aggregates. *Bulletin of earthquake engineering*, 10(1), 45-71.

- Carozzi, F. G., Poggi, C., Bertolesi, E., & Milani, G. (2018). Ancient masonry arches and vaults strengthened with TRM, SRG and FRP composites: Experimental evaluation. *Composite Structures*, 187, 466-480.

- Casarini, F., & Modena, C. (2008). Seismic assessment of complex historical buildings: application to Reggio Emilia Cathedral, Italy. *International Journal of Architectural Heritage*, 2(3), 304-327.
- Castellazzi, G., D’Altri, A. M., de Miranda, S., Chiozzi, A., & Tralli, A. (2018). Numerical insights on the seismic behavior of a non-isolated historical masonry tower. *Bulletin of Earthquake Engineering*, 16(2), 933-961.

- Cavalagli, N., Gusella, V., & Severini, L. (2017). The safety of masonry arches with uncertain geometry. *Computers & Structures*, 188, 17-31.

- Chen, X., Wang, H., Chan, A. H., & Agrawal, A. K. (2019). Dynamic failure of dry-joint masonry arch structures modelled with the combined finite–discrete element method. *Computational Particle Mechanics*, 1-12.

- Chiarugi, A., Fanelli, M., & Giuseppetti, G. (1993). Diagnosis and strengthening of the Brunelleschi Dome. *IABSE REPORTS*, 441-441.

- Chioccarelli, E., & Iervolino, I. (2010). Near-source seismic demand and pulse-like records: A discussion for L’Aquila earthquake. *Earthquake Engineering & Structural Dynamics*, 39(9), 1039-1062.

- Code, P. (2005). Eurocode 2: Design of concrete structures-part 1–1: General rules and rules for buildings. British Standard Institution, London.

- Code, P. (2005). Eurocode 6: Design of masonry structures-Part 1-1: General rules for reinforced and unreinforced masonry structures. Brussels: European Committee for Standardization.

- Code, P. (2005). Eurocode 8: Design of structures for earthquake resistance-part 1: general rules, seismic actions and rules for buildings. Brussels: European Committee for Standardization.

- Croci, G. (1995). The Colosseum: safety evaluation and preliminary criteria of intervention. *Structural Analysis of Historical Constructions, Barcelona*.

- Cusano, C., Cennamo, C., & Angelillo, M. (2019). Seismic vulnerability of domes: a case study. *Journal of Mechanics of Materials and Structures*, 13(5), 679-689.

- D’Ayala, D., & Speranza, E. (2003). Definition of collapse mechanisms and seismic vulnerability of historic masonry buildings. *Earthquake Spectra*, 19(3), 479-509.
- Dogangun, A., & Sezen, H. (2012). Seismic vulnerability and preservation of historical masonry monumental structures. *Earthquake and Structures*, 3(1), 83-95.

- Doglioni, C., Mongelli, F., & Pieri, P. (1994). The Puglia uplift (SE Italy): an anomaly in the foreland of the Apenninic subduction due to buckling of a thick continental lithosphere. *Tectonics*, 13(5), 1309-1321.

- FEMA (1992). NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings, FEMA 172, Federal Emergency Management Agency, Washington, D.C., USA.

- FEMA (1997). NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings, FEMA 274, Federal Emergency Management Agency, Washington, D.C., USA.

- FEMA (1998). Evaluation of earthquake damaged concrete and masonry wall buildings, FEMA 306, Federal Emergency Management Agency, Washington, D.C., USA.

- FEMA (1999). Repair of earthquake damaged concrete and masonry wall buildings, FEMA 308, Federal Emergency Management Agency, Washington, D.C., USA.

- FEMA (2009). Unreinforced Masonry Buildings and Earthquakes Developing Successful Risk Reduction Programs, FEMA P-774, Federal Emergency Management Agency, Washington, D.C., USA.

- Formisano, A., & Marzo, A. (2017). Simplified and refined methods for seismic vulnerability assessment and retrofitting of an Italian cultural heritage masonry building. *Computers & Structures*, 180, 13-26.

- Galassi, S., Misseri, G., Rovero, L., & Tempesta, G. (2018). Failure modes prediction of masonry voussoir arches on moving supports. *Engineering Structures*, 173, 706-717.

- Garofalo, V. (2016). The Geometry of a Domed Architecture: A Stately Example of Kārbandi at Bagh-e Dolat Abad in Yazd. *Nexus Network Journal*, 18(1), 169-195.

- Gattulli, V., Antonacci, E., & Vestroni, F. (2013). Field observations and failure analysis of the Basilica S. Maria di Collemaggio after the 2009 L’Aquila earthquake. *Engineering failure analysis*, 34, 715-734.
- Giuffrè, A. (1991). Readings on Mechanics of Historic Masonry. In Italian: Letture sulla Meccanica delle Murature Storiche, Kappa.

- Giuffrè, A. (1993). Safety and conservation of historical centers: The case study of Ortigia, In Italian: Sicurezza e conservazione dei centri storici: Il caso Ortigia, Editrice Laterza, Bari.

- Giuffrè, A. (1996). A mechanical model for statics and dynamics of historical masonry buildings. In Protection of the architectural heritage against earthquakes (pp. 71-152). Springer, Vienna.

- Haj Ghasem, K. (2004). The second section of the bazaar buildings, Ganjnameh of Islamic architecture of Iran.

- IASS WG 8. (1984). Analysis, Design & Construction of Space Frames. Bulletin of IASS XXV, 1/2, 85/85.

- Indirli, M., S. Kouris, L. A., Formisano, A., Borg, R. P., & Mazzolani, F. M. (2013). Seismic damage assessment of unreinforced masonry structures after the Abruzzo 2009 earthquake: The case study of the historical centers of L'Aquila and Castelvecchio Subequo. International Journal of Architectural Heritage, 7(5), 536-578.

- International Code Council ICC (2003). International Building Code, Reston, VA.

- Issue No. 120 (2003). “Iranian Concrete Code (ABA)”, Iranian Management Organization, 6th Edition, Tehran, Iran.

- Kaushik, H. B., Rai, D. C., & Jain, S. K. (2007). Stress-strain characteristics of clay brick masonry under uniaxial compression. Journal of materials in Civil Engineering, 19(9), 728-739.

- Kawaguchi, M. (2016). How well do we appreciate the structural legacies? In Proceedings of IASS Annual Symposia (Vol. 2016, No. 11, pp. 1-10). International Association for Shell and Spatial Structures (IASS).
- Kmiecik, P., & Kamiński, M. (2011). Modelling of reinforced concrete structures and composite structures with concrete strength degradation taken into consideration. *Archives of civil and mechanical engineering*, 11(3), 623-636.

- Kohrangi, M., Vamvatsikos, D., & Bazzurro, P. (2019). Pulse-like versus non-pulse-like ground motion records: Spectral shape comparisons and record selection strategies. *Earthquake Engineering & Structural Dynamics*, 48(1), 46-64.

- Lagomarsino, S., & Podesta, S. (2004). Seismic vulnerability of ancient churches: I. Damage assessment and emergency planning. *Earthquake spectra*, 20(2), 377-394.

- Lagomarsino, S., & Giovinazzi, S. (2006). Macroseismic and mechanical models for the vulnerability and damage assessment of current buildings. *Bulletin of Earthquake Engineering*, 4(4), 415-443.

- Lagomarsino, S. (2012). Damage assessment of churches after L’Aquila earthquake (2009). *Bulletin of Earthquake Engineering*, 10(1), 73-92.

- Lourenço, P. B., Rots, J. G., & Blaauwendraad, J. (1995). Two approaches for the analysis of masonry structures: micro and macro-modeling. *HERON*, 40(4), 1995.

- Lourenço, P. B. (2002). Computations on historic masonry structures. *Progress in Structural Engineering and Materials*, 4(3), 301-319.

- Lourenço, P. B., & Roque, J. A. (2006). Simplified indexes for the seismic vulnerability of ancient masonry buildings. *Construction and Building Materials*, 20(4), 200-208.

- Lu, P. J., & Steinhardt, P. J. (2007). Decagonal and quasi-crystalline tilings in medieval Islamic architecture. *science*, 315(5815), 1106-1110.

- Majewski, S. (2003). The mechanics of structural concrete in terms of elasto-plasticity. *Publishing House of Silesian University of Technology*, Gliwice.

- Memarian, H., Islam, A., & Mousavian, F. (2014). 15th Century Contribution to the Study of Vaulted Structure in Iran. *Iran University of Science & Technology*, 24(1), 1-8.
- Mendes, N., Zanotti, S., & Lemos, J. V. (2020). Seismic performance of historical buildings based on discrete element method: An adobe church. *Journal of Earthquake Engineering*, 24(8), 1270-1289.

- Motro, R. (2009). An anthology of structural morphology. World scientific.

- Noor-E-Khuda, S., Dhanasekar, M., & Thambiratnam, D. P. (2016). Out-of-plane deformation and failure of masonry walls with various forms of reinforcement. *Composite Structures*, 140, 262-277.

- Nooshin, H. (1998). Space structures and configuration processing. *Progress in Structural Engineering and Materials*, 1(3), 329-336.

- Öztürk, Ş., Bayraktar, A., Hökelekli, E., & Ashour, A. (2019). Nonlinear structural performance of a historical brick masonry inverted dome. *International Journal of Architectural Heritage*.

- Panella, D. S., Tornello, M. E., & Frau, C. D. (2017). A simple and intuitive procedure to identify pulse-like ground motions. *Soil Dynamics and Earthquake Engineering*, 94, 234-243.

- Pantò, B., Cannizzaro, F., Caddemi, S., & Caliò, I. (2016). 3D macro-element modelling approach for seismic assessment of historical masonry churches. Advances in Engineering Software, 97, 40-59.

- Papadopoulo, A., & Jazanî, H. (1989). Mi‘mārī-i Islāmī: Islamic Architecture. Markaz-i nashr-i farhangi-RAJ.

- Pasticier, L., Amadio, C., & Fragiacomo, M. (2008). Non-linear seismic analysis and vulnerability evaluation of a masonry building by means of the SAP2000 V. 10 code. *Earthquake engineering & structural dynamics*, 37(3), 467-485.

- Pellegrini, D., Girardi, M., Lourenço, P. B., Masciotta, M. G., Mendes, N., Padovani, C., & Ramos, L. F. (2018). Modal analysis of historical masonry structures: Linear perturbation and software benchmarking. *Construction and Building Materials*, 189, 1232-1250.

- Pirnia, M. K., & Bozorgmehri, Z. (2006). Hendese Dar Memari (Geometry in Architecture). Tehran: Sazman-e Miras-e Farhangi-ye Keshvar, Iranian Cultural Heritage Organization, Tehran, (In Persian).
- Portioli, F., & Cascini, L. (2017). Large displacement analysis of dry-jointed masonry structures subjected to settlements using rigid block modelling. *Engineering Structures*, 148, 485-496.

- Ramos, L. F., & Lourenço, P. B. (2004). Modeling and vulnerability of historical city centers in seismic areas: a case study in Lisbon. *Engineering structures*, 26(9), 1295-1310.

- Roca, P. (2001). Studies on the structure of Gothic Cathedrals. *Historical Constructions*, 71-90.

- Roca, P., Cervera, M., & Gariup, G. (2010). Structural analysis of masonry historical constructions. Classical and advanced approaches. *Archives of computational methods in engineering*, 17(3), 299-325.

- Rots, J. G. (2001). Sequentially linear continuum model for concrete fracture. *Fracture mechanics of concrete structures*, 2, 831-840.

- Sarhangi, R. (1999). The Sky Within: Mathematical Aesthetics of Persian Dome Interiors. *Nexus Network Journal*, 1(1), 87-98.

- Sarhosis, V., De Santis, S., & de Felice, G. (2016). A review of experimental investigations and assessment methods for masonry arch bridges. *Structure and Infrastructure Engineering*, 12(11), 1439-1464.

- Scozzese, F., Ragni, L., Tubaldi, E., & Gara, F. (2019). Modal properties variation and collapse assessment of masonry arch bridges under scour action. *Engineering Structures*, 199, 109665.

- Secretary of the Interior. (1995) Standards for the Treatment of Historic Properties: With Guidelines for Preserving, Rehabilitating, Restoring & Reconstructing Historic Buildings, National Park Service, Washington D.C.

- Severini, L., Cavalagli, N., DeJong, M., & Gusella, V. (2018). Dynamic response of masonry arch with geometrical irregularities subjected to a pulse-type ground motion. *Nonlinear Dynamics*, 91(1), 609-624.

- Sevim, B., Bayraktar, A., Altunişik, A. C., Atamürtktör, S., & Birinci, F. (2011). Assessment of nonlinear seismic performance of a restored historical arch bridge using ambient vibrations. *Nonlinear Dynamics*, 63(4), 755-770.
- Sharbaf, A. (2006). Gereh va Karbandi (Gereh and Karbandi). Tehran: Sazman-e Miras-e Farhang-iye Keshvar, Iranian Cultural Heritage Organization, Tehran, (In Persian).

- Solan, B., Ettema, R., Ryan, D., & Hamill, G. A. (2020). Scour Concerns for Short-Span Masonry Arch Bridges. *Journal of Hydraulic Engineering*, 146(2), 06019019.

- Solan, B., Nowroozpour, A., Clopper, P., Watters, C., & Ettema, R. (2019, September). Scour-induced failure of masonry arch bridges: Causes and countermeasures. In E-proceedings of the 38th IAHR World Congress.

- Stach, E. (2010). Structural morphology and self-organization. *Journal of the International Association for Shell and Spatial Structures*, 51(3), 217-231.

- Vicente, R., Lagomarsino, S., Ferreira, T. M., Cattari, S., & da Silva, J. M. (2018). Cultural heritage monuments and historical buildings: Conservation works and structural retrofitting. In Strengthening and retrofitting of existing structures (pp. 25-57). Springer, Singapore.

- Wang, T., & Hsu, T. T. (2001). Nonlinear finite element analysis of concrete structures using new constitutive models. *Computers & structures*, 79(32), 2781-2791.

- Zampieri, P., Zanini, M. A., & Faleschini, F. (2016). Influence of damage on the seismic failure analysis of masonry arches. *Construction and Building Materials*, 119, 343-355.

- Zampieri, P., Zanini, M. A., Faleschini, F., Hofer, L., & Pellegrino, C. (2017). Failure analysis of masonry arch bridges subject to local pier scour. *Engineering Failure Analysis*, 79, 371-384.

- Zampieri, P., Cavalagli, N., Gusella, V., & Pellegrino, C. (2018). Collapse displacements of masonry arch with geometrical uncertainties on spreading supports. *Computers & Structures*, 208, 118-129.

- Zampieri, P., Simoncello, N., & Pellegrino, C. (2019). Seismic capacity of masonry arches with irregular abutments and arch thickness. *Construction and Building Materials*, 201, 786-806.