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Seismic isolation code developments and significant applications in Turkey

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ABSTRACT

There is a special need to preserve the functionality of critical structures, such as hospitals, under severe earthquakes. In this sense, seismic isolation technology serves as a vital design method for the protection of their functionality.

In Turkey, seismic isolation technology has been applied at an accelerated pace to new or retrofitted buildings and infrastructures for earthquake protection essentially after the 1999 Kocaeli Mw7.4 Earthquake. Several guidelines and a new official code are prepared to encourage and regulate the on-going applications.

To enable the post-earthquake functionality of hospitals, the Ministry of Health public private partnership program foresees to build health campuses with seismic isolation. As of 2017, 21 health projects are complete or under construction with total investment of more than USD 23 billion.

Following a general review of seismic isolation design, the essential features of the recent seismic isolation code are provided and compared with European, Japanese and US Codes.

After a brief survey of base isolated hospitals in the world, two examples of large scale hospitals with seismic isolation are provided.

The Basibüyük Training and Research Hospital in Istanbul, retrofitted with seismic isolation, encompasses 750 beds in 113.000 m^2 floor area and is the largest hospital in the world retrofitted with a seismic isolation system consisting of 688 lead rubber and 154 sliding bearings.

The newly built Adana Integrated Health Campus (City Hospital) has 430,000 m^2 floor area and houses 1500 beds. With an isolation system composed of 1552 triple curved surface friction sliders, the hospital is currently the largest base isolated hospital in the world.

1. Introduction

Earthquake is a threat to human lives and assets. Population growth and increasing urbanization in earthquake-prone areas suggest that earthquake impacts on human populations will continue in the coming decades.

Although, seismic design codes have been very successful in reducing collapse of structures, and have saved the lives of people, the same level of success is not seen in non-structural and business losses. In fact, in developed countries, over the past 20 years, most of the economic losses caused by earthquakes have resulted from non-structural damage and loss of facility use.

Modern buildings contain sensitive and costly equipment that are vital in business, commerce, education and health care. The contents of these buildings are generally more costly and valuable than the buildings themselves. Furthermore, hospitals, communication and emergency centers, and police and fire stations must be operational immediately after an earthquake, when the need is greatest. In connection with the “Performance Based Seismic Design” approach, the expected performance objective for such critical facilities should be "fully operational", under exposure to the design basis earthquake (DBE).

Conventional construction techniques may result in very high floor accelerations in stiff buildings and large inter-story drifts in flexible structures, causing difficulties in ensuring the safety of the non-structural components and contents. In order to achieve a "fully operational" performance, the most promising design approach is to use seismic isolation technique.

Seismic isolation allows for the installation of specially designed bearing (isolator) units at the foundation or any other convenient floor level to substantially decouple the superstructure from earthquake motions.

Increasing the fundamental period of vibration away from high spectral acceleration zone and the concentration of nonlinearity at the isolation interface serves to avoid the inelastic response of the superstructure and keeps the earthquake induced responses in the limited ductility level (Fig. 1). In addition, seismic isolators also reduce the floor accelerations and the inter-story displacements.

Although, contemporary seismic isolation technologies were first proposed as an innovative performance enhancement strategy from 1970s to 2000s, nowadays it is transformed to mature and arguably the best way of earthquake protection method. As of 2014, more than 23,000 structures, located in over 30 countries, have been so far

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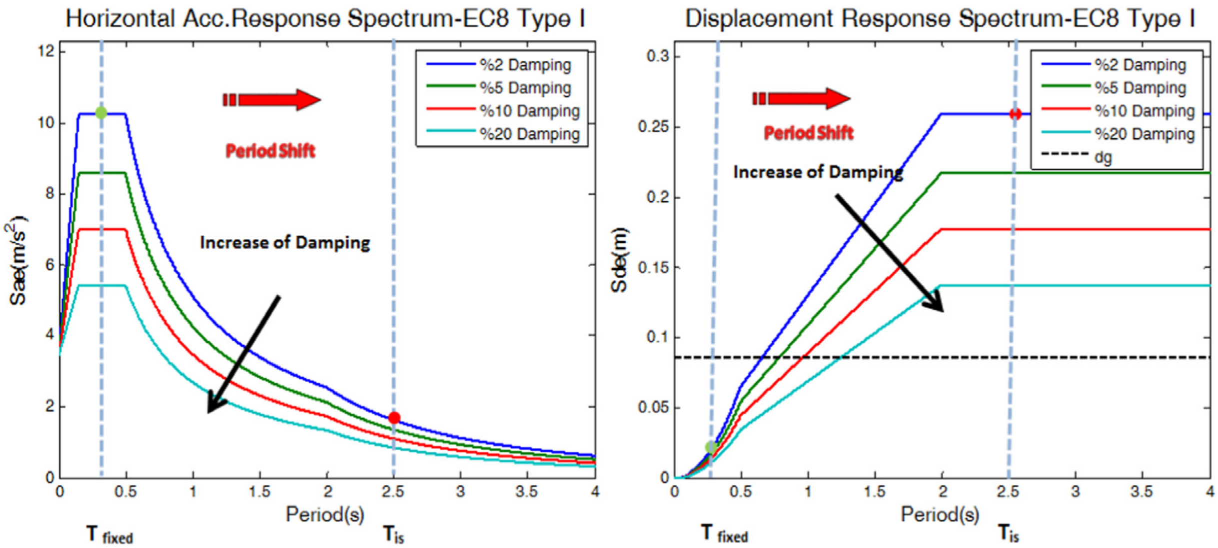


Fig. 1. Benefits of seismic isolation through the elongation of fundamental vibration period and limitation of excessive deformations with increase in damping (after [15]).

protected by passive anti-seismic systems, mainly by the seismic isolation [36]. Japan is the leading country for the overall number of applications, followed by China, Russia, Italy and USA.

Turkey is located in an earthquake prone region and suffered high amount of casualties and loss of property due to earthquakes over many centuries. Potential impacts of large earthquakes on urban societies need to be mitigated through multi-disciplinary approaches. In Turkey, seismic isolation technology has been applied at an accelerated pace to new or retrofitted buildings and infrastructures for earthquake protection essentially after the 1999 Kocaeli Mw7.4 Earthquake. As of 2017, there exist a multitude of structures with seismic isolation, including hospitals, schools, airport terminals, LNG storage tanks, highway and railway viaducts and stadia. Most of the recent activity have focused on viaducts and hospital buildings. To date, the numbers of structures constructed with seismic isolation devices is 72.

These fast developments on seismic isolation also necessitated the preparation of a national official design code for the seismic isolation applications on building.

Turkey, especially for large hospitals, is firmly committed to base-isolation methodology, since the health care facilities are expected to be functional and serve after a major seismic event. Notably, Turkey has embarked on a program to build numerous large hospitals complexes with seismic isolation [23,27,29].

Engineering News Record (ENR) has recently selected the largest 10 base isolated buildings in the world (https://www.enr.com/articles/42366-the-10-largest-base-isolated-buildings-in-the-world). The ranking was based on the total closed floor area. Three of these largest base isolated buildings are located in Turkey.

In summary, today, seismic isolation method is a justified, mature and reliable performance enhancement strategy for a wide range of structural systems. As a result of the targeted response modification, high performance expectations and earthquake resilience can be achieved during the service life of structures that are in compliance with the design code requirements.

2. Seismic isolation design

In general, conventional (i.e. fixed base) buildings designed in conformity with earthquake resistant design codes should: (1) Resist a minor level earthquake ground motion without damage; (2) Resist a moderate level earthquake ground motion without structural damage, but possibly with nonstructural damage and; (3) Resist a major level earthquake without collapse.

Performance objectives in building design codes differ for traditional fixed-base and seismically isolated structures. Design provisions for isolated buildings, aim to avoid the structural damages and limit the non-structural damages to ensure "immediate occupancy" performance level under exposure to a design earthquake ground motion.

In this regard, the EC-8 [21] allows a maximum behavior (or response modification) factor of 1.5, ASCE/SEI 7-16 (2016) allows the response modification factor to be 0.375 times that used for the corresponding fixed-base structure (however, capped at 2) and, the Japanese building code (BSL-2015) allows for only limited inelastic response.

The design earthquake ground motion in EC-8 [21] corresponds to a seismic action with probability of exceedance 10% in 50 years (i.e. 475 year average return period). However, an importance factor of 1.4 is assigned for vital or strategic buildings (e.g. hospitals) which implicitly increases the return period to about 2000 years and, furthermore, tests of isolation devices is made by multiplying the actions by a factor of 1.2. In ASCE/SEI 7–10 the design earthquake is taken as the 2/3 of the MCE_R (Risk-Targeted Maximum Considered Earthquake, approximately equal to 2475 year return period earthquake with deterministic caps), whereas in ASCE/SEI 7–16, the design earthquake is taken directly as the MCE_R ground motion. In Japanese building code the design basis ground motion is prescriptively defined and can be estimated to correspond to a 500 year average return period for life-safety limit design (Otani [42]).

Inter-story drifts and floor accelerations are two key parameters in the seismic design of structures to avoid excessive damages both in structural and drift-sensitive non-structural elements. The maximum drift ratio of the superstructure varies between different codes. In ASCE/SEI 7–16 (2016) it is limited to 1.5% of the story height, for response spectrum analysis, whereas, 2% is allowed in response history analysis. In EC8 [21] the maximum drift ratio is 0.5% to protect brittle non-structural elements (0.75% otherwise) in the damage limitation level design. In Japanese code (BSL-2015) the drift limits are set as 0.5% and 2% respectively for damage limitation and life safety level designs.

2.1. Direct displacement based design for seismic isolation

Almost all codes on seismic isolation design use a mixture of force- (or strength-) based design and displacement based procedures. It is widely recognized that the traditional force-based design cannot directly implement the concepts of performance-based earthquake

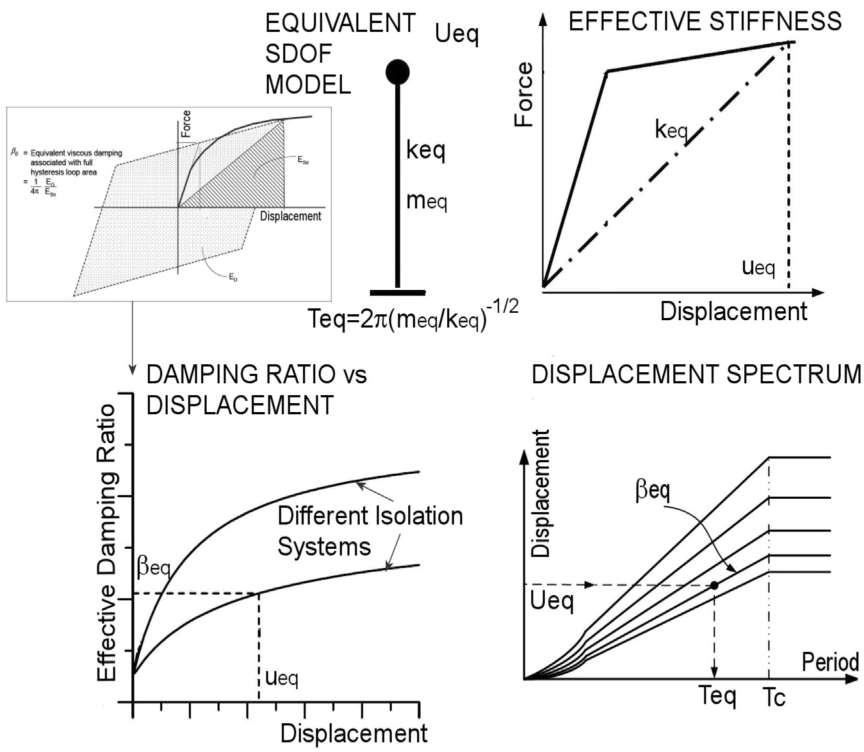


Fig. 2. Design steps of DDBD for SI structures (after [45]).

engineering since the performance (and/or damage) levels, are better correlated to displacements rather than forces [8,44]. In this connection, displacement-based design approaches have been proposed, such as the Direct Displacement-Based Design (DDBD), proposed by Priestley et al. [45].

The DDBD procedure begins with equating the displacement demand to the displacement capacity of the structure and iterations are required if equivalent damping is to be modelled with good precision. The displacement spectrum, which represents the seismic demand for the estimated level of damping, is then used in conjunction with the force-displacement relationship (capacity curve) of the structure to calculate the strength demand. In the DDBD procedure the nonlinear MDOF model of the actual building is replaced by an equivalent linear SDOF system (Substitute Structure, originally proposed by [46]), whose properties correspond to the effective lateral stiffness and equivalent viscous damping of the real structure at the peak displacement response associated with the given level of seismic excitation.

The Equivalent Lateral Load (ELF) procedure, sanctioned in current design codes as a simple method of analysis, demonstrates the importance of iterative DDBD method either in the determination of seismic demands for the preliminary design phase and, consequently, in the establishment of minimum isolator displacement and base shear levels.

The key components of the DDBD process for seismic isolation are indicated in Fig. 2. The essential ingredients of this process are: (1) Creation of the equivalent SDOF model; (2) Effective stiffness, which is obtained from the force versus displacement diagram (hysteresis curve); (3) Relationship between effective damping ratio and the displacement and; (4) Displacement spectrum for different effective damping ratios. The second and the third components are controlled by the relevant characteristics of the isolation system and the displacement spectrum is determined by the site-specific ground motion characteristics.

Displacement spectrum is one of the main ingredient of displacement-based design procedures. ASCE/SEI 7-16 (2016) and EC8 [21] define the displacement spectra as illustrated in Fig. 3. The transition period in the long period region vary between 1.2 s and 2 s in EC8 [21] and 4–16 s in ASCE/SEI 7–16 (2016). Needless to say, the near-fault (rupture directivity) and the directionality (i.e. conversion from “geometric mean” ground motion “maximum direction” ground motion) effects are important modifiers for the displacement spectra and should constitute an essential part of the design basis ground motion assessment.

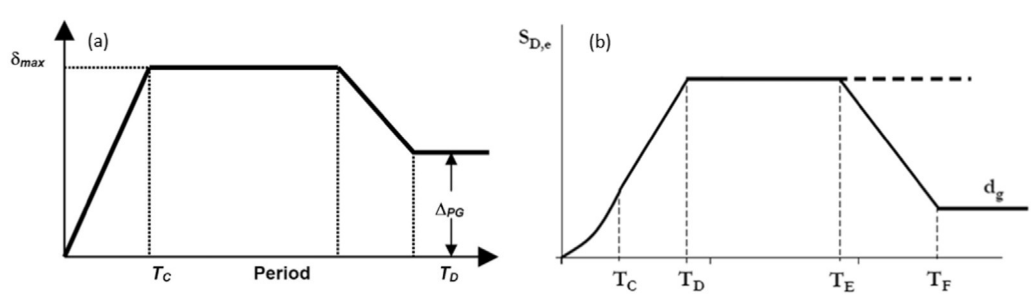


Fig. 3. (a) ASCE/SEI 7–10 displacement spectrum (b) Eurocode-8 displacement spectrum (after [25]).

3. Turkish seismic isolation design code for buildings

In Turkey, since 2000, academicians, design professionals and regulators have recognized passive control technologies as a viable mean to limit the earthquake-induced damage. The lack of the official seismic isolation design code have led professional engineers to the use the US (ASCE 7-5, 2005 and [3]) and European (EC8 [21]) codes for the seismic isolation design for buildings. The different approaches and procedures in these codes, especially in the design basis ground motion definitions, have unfortunately led to non-uniform applications. The testing of the isolator bearings have also followed these codes (and even their mixtures), thereby causing test results incompatible with the design parameters used.

In response to the advent of seismic isolation technology and increased number of seismic isolation applications in Turkey, a guideline entitled “Seismic Isolation Design Code for Buildings” was first published in 2008 [47]. ISMEP (Istanbul Seismic Risk Mitigation and Emergency Preparedness Project, www.ipkb.gov.tr) adopted this guideline in the applications for the seismic isolation design of new and retrofitted hospitals.

The total number of seismic isolation applications in Turkey, especially for hospitals, has significantly increased after the 1999 Kocaeli (Mw 7.4) and Düzce (Mw 7.2) earthquakes, where, 12 hospital buildings (26% of out of the 47 hospitals) were damaged beyond repair [30]. Furthermore, 28 health centers were totally destroyed while 20 others were heavily damaged. In 2011 Van, Turkey, earthquake, several hospitals were damaged (structural, non-structural and equipment) in the cities of Van and Ercis [20,31]. Due to severely limited hospital capacity, temporary field hospitals were deployed in the region. Yüzüncü Yil University Hospital, Maternal Health and Pediatric Hospital, Ipekyolu State Hospital and Ercis State Hospital were closed (or partially operating) after the earthquake due to heavy non-structural damage.

These problems on the post-earthquake operation of the hospitals have led Turkish Ministry of Health to release a regulation and technical specifications in 2013 to enforce that “Hospital Buildings, located in seismic zones 1 and 2 with number of bed capacity over 100 should be constructed with base-isolation”.

The Turkish Ministry of Health has also prepared a technical specification for the seismic isolation design of hospital buildings. The basic design criteria and the performance objectives of the seismic isolation design specifications prepared by Ministry of Health can be summarized as follows:

- The structural system should be designed based on an earthquake ground motion level associated with a probability of exceedance 10% in 50 years. The structural system below the isolation layer shall be designed to remain essentially elastic, in other words the response modification factor shall be taken as 1.0. The structural system above isolation layer shall be designed by using a response modification factor of 1.5, at maximum.
- The isolation system displacement shall be determined for an earthquake ground motion level associated with a 2% probability of exceedance in 50 years. The isolation system displacements shall be increased by 10% (at minimum) to account for the torsional effects.
- The story drift values above isolation layer shall be less than 0.50%.
- The base shear transmitted to the superstructure shall be less than 20% of the superstructure seismic weight at the earthquake ground motion level associated with 10% probability of exceedance in 50 years.
- The average horizontal acceleration at each story above isolation level should be less than 0.20 g (increased to 0.3 g in a recent revision of the specification).
- The non-structural elements (i.e. heating, ventilating, and air conditioning, plumbing, electro-mechanic systems) crossing the isolation layer, should be designed to accommodate the isolation system displacement and to continue its function after earthquake.

Table 1
Seismic Design Categories.

DBE level S_{DS} values	Seismic Design Category (SDC, enhanced and normal)	
	Building Risk Category = 1	Building Risk Category = 2, 3
$S_{DS} < 0.33$	SDC = 4a	SDC = 4
$0.33 <= S_{DS} <= 0.5$	SDC = 3a	SDC = 3
$0.50 <= S_{DS} < 0.75$	SDC = 2a	SDC = 2
$S_{DS} >= 0.5$	SDC = 1a	SDC = 1

In 2017, a code on Seismic Isolation Design for Building Structures is prepared by a committee consisting of academicians and professional engineers, in connection with the current revision of the Turkish Earthquake Resistant Design Code (which will be referred as: TERDC [51]. This code will be officially enforced in January 1, 2019.

The Turkish seismic isolation code is essentially based on the principles of ASCE 7-10 [3] and ASCE 7-16 [6] with some influences from the EC8 [21]. In addition ASCE 41-13 [5], Constantinou et al. [16] and McWitty and Constantionou [39] are considered.

The materials used in the manufacturing process of elastomeric isolation units are stipulated to conform to the requirements of the Chapter 8.2.2 of the European Standard (EN 15129 [22]) and, unless otherwise stated in the code, the design of the isolation units will be based on European Standard EN 1337-3 (2005).

For seismic isolation design 475-year (DBE-Design Basis Earthquake) and 2475-year (MCE-Maximum Considered Earthquake) ground motion levels are used.

The revised earthquake hazard map, that will be associated with the TERDC [51] provides the short period (S_s) and 1 s period (S_1) spectral acceleration parameters for average return periods of 43, 72, 475 and 2475 years [17].

TERDC [51] considers different levels of design, to meet different "performance levels" at different earthquake ground motion levels. These performance levels are: "Continued Functionality"(CF.); "Limited Damage / Immediate Occupancy" (LD); "Damage Control / Life-Safety" (DC) and; "Collapse Prevention" (CP).

The code encompasses four normal and four enhanced Seismic Design Categories (SDC) that depend on the Building Risk Categories and level of site-specific short period spectral acceleration (S_{DS}). Building Risk Categories measure the potential for consequential human casualty in three building use classes, that ranges from high risk (essential and critical buildings –Building Risk Category-I) to low risk (ordinary residential buildings-Building Risk Category-III). Table 1 shows the Seismic Design Categories (SDC) for given DBE level S_{DS} values and Risk Classes.

Table 2 provides the minimum performance levels and the permitted design procedure to be considered in seismic isolation design. SBD and DBD abbreviations stand, respectively, for Strength Based Design and Displacement Based Design.

The contents of the seismic isolation code for building structures encompass:

Mechanical Properties of the Isolation Devices
  Elastomeric Bearings
  Properties of the Elastomeric Isolation Units
  The Variation of the Mechanical Properties of Elastomeric Isolation Units
  Connection of Elastomeric Isolation Units
  Curved Surface Slider Type Isolators
  Properties of the Flat and Curved Surface Slider Type Isolators
  Design Essentials of the Flat and Curved Surface Slider Type Isolators
  The Variation of the Mechanical Properties of Curved Surface Sliding Isolator Units

Basic Design Principles of the Isolation System
- Fundamental Properties of the Isolator Units
- Properties of the Ground Motion
- Fundamentals of Design of Isolation System
- Lateral Restoring Force
- Modeling of Isolated Buildings

Earthquake Ground Motion
- Ground Motion Levels
- Earthquake Design Spectra

Seismic Analysis Methods
- Analysis Methods and Properties
- Equivalent Lateral Load Method
- Mode Superposition Method
- Linear Response History Analysis
- Nonlinear Response History Analysis

Peer Review Board

Tests of Isolation Units
- Prototype Tests
- Production Tests

Important points of the Turkish seismic isolation code are indicated below:

3.1. General

- Every stage of seismic isolation design conducted under this Code shall be controlled and approved by the peer review board.
- For buildings, encompassed in this Code, the isolation system composed of isolation devices should be placed in an isolation interface located under the main body of the building
- Buildings seismically isolated and designed according to this Code should remain functional with no damage in structural and non-structural elements at the design level earthquake. Similarly, Buildings seismically isolated and designed according to this Code the structural system should receive no damage and the isolation system should be stable at the maximum considered earthquake level.
- The isolation system must have the properties of: High vertical stiffness; Low lateral stiffness; Ability to carry vertical loads; Energy absorption capacity; Ability to re-center after seismic motion and; Adequate lateral stiffness against lateral forces (e.g. wind force) other than earthquake.
- In order to decrease the torsional effect on the system, the projections of the effective center of rigidity of the isolation system and center of mass of the superstructure on the isolation interface must be as close as possible.
- In the design of the isolation system, avoidance of tensile forces in the isolation devices is essential. In specific cases, special devices that can accommodate tension can be used. The functionality of such devices should be ensured by laboratory tests.
- The superstructure and the substructure can be designed according to the prescriptions for limited ductility.
- Nominal values of the isolation system properties can be used for the determination of floor responses.

3.2. Design

- In the design process two levels of earthquake shall be taken into consideration: Design Basis Earthquake (DBE) Ground Motion Level: Site dependent ground motion with 10% probability of exceedance in 50 years that corresponds to the average return period of 475 years. Maximum considered Earthquake (MCE) Ground Motion Level: Site dependent ground motion with 2% probability of exceedance in 50 years that corresponds to the average return period of 2475 years.
- The design of the isolation units will be based on "European Standard EN 1337–3:2005: Structural Bearings - Elastomeric Bearings", for the provisions that are not encompassed in this Code.

Table 2
Performance Levels and the Permitted Design Procedure.

New Buildings with Seismic Isolation (Super-structure)				
Earthquake Level	Seismic Design Category: 1–4, 3a, 4a		Seismic Design Category: 1a, 2a	
	Normal Performance	Permitted Design Procedure	High Performance	Permitted Design Procedure
DBE	LD	SBD	CF	SBD
MCE	–	–	–	–
Existing Buildings Retrofitted with Seismic Isolation (Super-structure)				
Earthquake Level	Seismic Design Category: 1–4, 3a, 4a		Seismic Design Category: 1a, 2a	
	Normal Performance	Permitted Design Procedure	High Performance	Permitted Design Procedure
DBE	DC – LD[1]	SBD	LD – CF[1]	SBD
MCE	–	–	–	–
New and Retrofitted Buildings with Seismic Isolation (Isolation System and Sub-Structure)				
Earthquake Level	Seismic Design Category: 1–4, 3a, 4a		Seismic Design Category: 1a, 2a	
	Normal Performance	Permitted Design Procedure	High Performance	Permitted Design Procedure
DBE	–	–	–	–
MCE	CF	DBD[2] – SBD[3]	CF	DBD[1] – SBD[2]

[1]For Risk Category "I" Buildings [2] For isolation system [3] For sub-structure.

- The upper and lower bound properties of the isolator properties shall be obtained by multiplying the nominal values with the maximum and the minimum property modification factors.
- For each isolator unit type, the maximum and the minimum property modification factors are obtained in terms of the property modification factors for "aging and environmental effects", "testing conditions" and "manufacturing variations".
- For all load combinations, the factor of safety against global overturning at the isolation interface shall be greater than 1.0.
- Response Modification Coefficients of 1.2 and 1.5 are stipulated respectively for the "continued functionality" (CF) and "limited damage / immediate occupancy" (LD) performance levels.
- The isolation system shall be configured to produce a lateral restoring force at any displacement up to its maximum design displacement. To ensure re-centering, the restoring force of the isolation system at the MCE level displacement shall be greater than the restoring force respectively at half of this displacement by not less than 2.5% of the seismic weight of the superstructure.
- To ensure stability, the uncoupled vibration period of the isolated building computed by using the post-elastic (2nd) stiffness of the idealized bi-linear response of the isolation system shall be less than 6 s.
- The analysis methods listed below shall be used depending on the properties of the building and isolation system: (a) Equivalent Lateral Force (ELF) Method, (b) Mode Superposition Method and, (c) Nonlinear Response History Analysis (NLRHA)
- Equivalent Lateral Load Method can be used if the specific requirements are satisfied. Mode superposition and nonlinear response history methods can be used where equivalent lateral load method is not applicable. Nonlinear response history analysis can be used in any case.
- Mode Superposition Method can be used if the effective damping ratio is less than 30%.
- Maximum internal forces and displacements in Mode Superposition Method shall be determined by a statistical combination of contribution of sufficient number of natural vibration modes.
- If the isolation system cannot be modelled as equivalent linear, the use of Nonlinear Response History Analysis is mandatory. For the application of nonlinear response history method the following is in order:

(a) The nonlinear hysteretic behavior of the isolating system that accounts for its dependence on strain rate, vertical load and bi-directionality of motion should be explicitly modelled in nonlinear response history analysis. Nonlinearity in the sub- and superstructure does not need to be considered.
(b) A minimum eleven sets of earthquake ground motions (acceleration records with two orthogonal horizontal and one vertical component) with the following properties shall be selected for the analysis to be performed in the time domain.
(c) If the uncoupled vertical vibration period of the isolated building is larger than 0.1 s, the vertical degree of freedoms shall be considered in the sub- and super-structure models and both the vertical and horizontal component of the ground motion shall be taken in to account in the design process.
(d) For cases where the soil class is weaker that ZD (NEHRP soil class D) the design process should incorporate 3D soil-structure interaction analysis.

- Isolation system can be modelled as equivalent linear if the following requirements are satisfied;

(a) The ratio of the equivalent linear (secant) stiffness of the isolation system corresponding to the design displacement to the equivalent linear (secant) stiffness corresponding to 20% of the design displacement shall be at least ½,
(b) For elastomeric isolators, the properties of the isolation unit at design displacement shall differentiate at most 10% depending on the vertical loading and,
(c) The equivalent damping ratio of the isolation system at DBE and MCE levels shall not exceed 30%.

- The inter-story drift ratio of each story shall be less than 0.005 and 0.01 respectively for Continued Functionality (CF) and Limited Damage (LD) performance levels.

3.3. Equivalent lateral force (ELF) method

- The equivalent lateral force (ELF) method is the basic analysis method and will be used for the initial design of the isolated building and, sizing of the isolator units and to provide reference design values. ELF method uses the maximum direction response spectra that requires the modification of the geometric mean (GeoMean) response spectra used in the general earthquake resistant design. As such, the displacement at the center of rigidity of the isolation system at DBE and MCE design level earthquakes is computed with spectral acceleration values multiplied by the factor 1.3.
- The Spectrum Modification Factor is taken as: $\eta = [10/(5+\xi_{\mathrm{eff}})]^{1/2}$ (where ξ_{eff} is the effective damping ratio at DBE and MCR displacement levels)
- Equivalent Lateral Force (ELF) method can be used once the requirements listed below are satisfied;

(a) Building must be located on ZA, ZB, ZC or ZD (corresponds to NEHRP A, B, C and D soil classes) type of soil.
(b) Effective damping ratio less than 30%.
(c) The superstructure have only limited irregularity.
(d) Number of stories and the total height of the superstructure must be less than, respectively, 4 stories and 20 m (to limit the effect of higher modes of vibration).
(e) The period of vibration the isolated building at the MCE level must be less than 4 s.
(f) The uncoupled vertical period of vibration of the isolated building should be less than 0.1 s.
(g) No uplift or tension on the isolation units.

- In the ELF method, the contribution of the vertical component of the earthquake ground motion on the vertical column loads shall be computed by multiplying the seismic mass of the superstructure with two-thirds of the horizontal short period spectral acceleration.
- In the ELF method, the total isolation unit displacements cannot be less 1.1 times the displacement obtained at the center of rigidity of the isolation system.
- Equivalent lateral shear force acting on the isolation system at design earthquake level displacement shall be determined by the multiplication of the upper bound effective stiffness and the displacement of the center of rigidity of the isolation system at DBE level.
- Total equivalent seismic force shall be greater than the shear force due to wind design loads and the effective yield force of the isolation system.
- To implicitly account for the contribution of higher modes of vibration, the total equivalent seismic shear force shall be distributed to the story levels by an inverted triangular height-wise distribution.
- ELF analysis needs to be considered for all cases, regardless of its applicability, since the ELF results provide the following bounds to the other two methods.

(a) The lower limit to the design base shear in the superstructure from a Mode Superposition Method analysis is 90% of that from an ELF analysis if the superstructure is irregular or 80% of that if it is regular.
(b) The lower limit to the design shear below the isolation system provided by a Nonlinear Response History Analysis is 90% of that from an ELF approach.
(c) The lower limit to the total displacement from a Mode Superposition Method or a Nonlinear Response History Analysis under the design seismic action and the MCE are 90% and 80%, respectively, of the value from the ELF method.
(d) All force results from Mode Superposition Method or a Nonlinear Response History Analysis must be re-scaled upwards to match the base shear lower limits (ELF results).

3.4. Tests

- The force–displacement characteristics, effective damping ratio, effective horizontal and vertical stiffness of the isolation units of the isolation system shall be determined by tests and verified with the values used in the design process.
- Prototype Tests involve: Long duration compression test; Vertical stiffness test; 20 cycle service test; Series of combined compression and shear cyclic tests to establish effective stiffness and damping at DBE and MCE level displacements; Cyclic load test to check durability; Ramp test to verify isolator stability and; Uplift / tension test (if needed).
- Production Tests involve: Long duration compression test and combined compression and shear cyclic test at DBE level.
- For all tests the specimen shall remain stable and slope of the force-deformation curve shall always remain positive.
- Detailed acceptance rules are stipulated based on the comparison of results over the cycles of tests and the difference of the upper and lower bound values of the isolator properties with the range obtained from the test results.

3.5. Comparison of Turkish code with other codes

Seismic design codes and specifications for seismic isolation, in particular, Japanese Code [11], USA Code ASCE-7-10 (2010) and ASCE 7-16 [6], Italian Code NTC-08 [41] representing the EC8 [21], and the 2018 revision of the Turkish Seismic Design Code [51] will be compared with emphasis on the basic design procedures.

In Japan, the performance based design concepts have been taking effect since 1998 in the Building Standard Law and it has been associated with the Enforcement Order [11]. Notification No. 1457 (2009) of the Minister of Land, Infrastructure and Transport cover the design of seismically isolated structures, whereas the quality and general features of the isolation devices are governed by the Notification No. 1446 (Otani and Kani, 2002 [43]). The "Japanese Society of Seismic Isolation (JSSI) Standard for Devices," which was issued in 1997 have been used as a reference document in the establishment of Notification No.1446. The performance based design concepts are implemented in the two-stage (Level-1 and Level-2) design levels. Level-1 represents the damage limitation performance objectives for serviceability limit state with 50 years return period whereas the Level-2 is the life safety limit state with approximately 500 years return period. For the design three choices (routes) are articulated. Route-1 is intended for small buildings and requires no computation. Route-2 is applicable to ordinary/normal buildings and essentially follows the ELF procedure. Route-3 encompasses nonlinear time history analyses.

In USA, the seismic isolation regulations in ASCE/SEI 7 standards has evolved over the years (starting with the 1991 Uniform Building Code) to implement the developments in the design and method of analysis. The latest version of the standard [6] introduces significant changes over ASCE 7-10 [3]. Such as: the design is based on MCE_R (Risk Targeted Maximum Considered Earthquake) ground motion level only; ELF procedure is applicable to a much wider range of projects; the vertical distribution of lateral forces is based on a rational procedure and; the property modification factors are systematically determined. It should also be noted that in ASCE/SEI 7 standards, the definition of horizontal ground motion has changed from the geometric mean (GeoMean) of spectral acceleration components to the peak response (i.e. maximum direction) of a SDF system.

In Italy, the design of buildings with seismic isolation follow the related section of NTC-08 [41], in compliance with Eurocode-8 (CEN, 2005) and EN-15129 [22]. There exist two main design limits. The "no-collapse" Ultimate Limit State (ULS), is referred to a design seismic action with return period of 475 years with an importance factor of 1.4 (2 for critical buildings). The Damage Limit State (DLS) is checked with respect to the inter-story drift limits. Nonlinearity is concentrated at the isolation level, the substructure remains essentially elastic (behavior factor $q = 1$) and the superstructure encompasses sufficient amount of rigidity and strength (behavior factor $q < = 1.5$). For the analysis and design of isolated structures three methods of analysis namely ELF, linear dynamic analysis and nonlinear time history analysis are offered in the Italian code.

All of these codes introduce essentially three procedures for the analysis and design of seismically isolated structures: equivalent lateral force, modal superposition and non-linear response history analysis/design. Among these three methods, the Equivalent Lateral Force (ELF) procedure represents the simplest, yet basic, procedure that incorporates the equivalent linearization of nonlinear isolation system in a static design approach. For isolated bridge design, this procedure is referred as Uniform Load Method (ULM) in the AASHTO Guide Specifications for Seismic Isolation Design [1]. The inherent similarity of the ELF procedure to the Direct Displacement Based Design (DDBD) was indicated in Section 2.1.

Even though, ELF procedure is defined as the simplest method of analysis, it has a critical role from the preliminary design phase to the final design stage by providing important insight about the system behavior and controlling the key demand parameters needed for the design [50].

In the ELF method, calculation of demand parameters is based on assumption of a rigid superstructure, amplitude dependent effective (or secant) stiffness, K_{eff} and viscous damping ratio, β_{eff}, properties of the entire isolation system. Based on the single degree of freedom (SDOF) model, the displacement dependent effective period, T_{eff}, of the isolated building is calculated on the basis of K_{eff} and the seismic weight of the superstructure. β_{eff} is an important value to determine spectrum adjustment factor on the basis of equivalent viscous damping assumption.

The lateral displacement of the isolation system and the associated base shear force are the most critical parameters estimated by the ELF procedure. Similar to the DDBD summarized in Section 2.1, the calculation of the displacement with respect to ELF method is an iterative process depending on the effective stiffness and effective damping values. Iterations start with the assumed value of displacement and end up with a displacement value calculated sufficiently close to the assumed one. NTC-08 [41], ASCE/SEI 7-10 and 7-16 (2010 and 2016), BSL (2016) and TERDC [51] codes utilize similar procedures for the ELF method. One of the most important differences between these codes is the representation of the seismic demand. Tables 3, 4 (modified after [50]) provide for a critical comparison of ASCE/SEI 7–16 (2016), BSLEO (2016), NTC-08 [41] and TERDC [51] codes in terms of ELF method of analysis/design.

4. Seismic isolation applications for hospitals

As of 2014, more than 23,000 structures, located in over 30 countries, have been so far protected by passive anti-seismic systems, mainly by the seismic isolation [36]. Japan is the leading country for the overall number of applications, followed by China, Russia, Italy and

Table 3
Comparison of Essential Features of Seismic Isolation Codes.

Code	ASCE/SEI 7–16	[11]	NTC-08	[51]
Design methods	ELF/RSA/NLRHA	No Calculation /ELF /NLRHA	ELF/RSA/NLRHA	ELF/RSA/NLRHA
Return Period (year)	MCE_R (2475)	50/500 (Estimated)	475 - supestrucure[a] 975- isolation system[a]	475/2475 DBE/MCE
Safety factor on Isolation capacity	From tests, Implicit in MCE_R	Elastomeric = 0.8 Sliding/Friction = 0.9	From tests, γ_x = 1.2 (Reliability)	From tests, Implicit in MCE
Design Requirements	Low ductility ($R \leq 2$) $V_{base} = V_{ELF}$	Elastic $V_{base} = 1.3\ V_{ELF}$	Low ductility ($q \leq 1.5$) $V_{base} = V_{ELF}$	Low ductility ($R \leq 1.2/1.5$) $V_{base} = V_{ELF}$
Modeling	2D for ELFM, otherwise 3D	Simple 2D, even for NLTHA	2D for ELFM, otherwise 3D	2D for ELFM, otherwise 3D
Drift Ratio	1:50	1:200 / 1:50	1:200	1:200 / 1:100
Location of Devices	–	Base Only	–	Base Only
Kv/Ke	–	–	≥ 800	–
Tension in Isolators	Allowed	Not Allowed	Not Allowed	Not Allowed
Max. Vertical Period (T_v) for vertical analysis	–	–	$T_V < 0.1$ s	$T_V < 0.1$ s
Importance Factor	–	Based on request of control mechanism	EC8 [21] = 1.4, NTC – 08 V_R (V_N, C_U)	–

[a] For a standard buildings.

USA. As of 2011 the number of base isolated large buildings (i.e. excluding houses) in Japan, amounted to 1100. 12% (132) of this number were hospitals [36].

Some of the important worldwide applications of new hospital constructions with seismic isolation are as follows.

Opened in 1991 as the first base-isolated hospital in USA, the University of Southern California (USC) hospital building experienced the 1994 Northridge earthquake. The eight-story 40,000 m^2 hospital building has a concentrically braced steel frame supported on 68 lead rubber and 81 elastomeric isolators. The base-isolated hospital building performed well, the superstructure remained elastic due to the effectiveness of base isolation and significantly reduced the response when compared to a fixed-base structure [40].

Inagi City Hospital, housing 290 beds, is the first public hospital adopting seismic isolation system in Japan, built in 1998.

Built in 1998, Frosinone's Hospital, Italy, was reported to be the first hospital in Europe with seismic isolation [34].

The 414-bed Arrowhead Medical Center located in Colton, California, built in 1993, was reported to be the largest base isolated hospital buildings in the world [19]. The total floor area 86,400 m^2 and the isolation system consists of 392 high-damping rubber isolators and 184 fluid viscous dampers ([12]; Asher et al. [7]).

Del Mare Hospital, in Naples, Italy, has a plan layout of about 150 × 150 m and the total height is about 32 m. The seismic isolation system encompasses 327 high damping rubber bearings. Del Mare Hospital has been reported as the largest base isolated hospital in Europe [13,19].

New Stanford Hospital is 7-story steel moment frame structure isolated with 206 triple friction pendulum bearings at the base. Construction of the $5-billion new facility encompasses 600 beds in almost 90,000 m^2 floor space (http://www.sumcrenewal.org/projects/project-overview/stanford-hospital/).

In New Zealand, Christchurch Hospital Acute Services Building (a 62,000 m^2 10 story structure) is under construction with a seismic isolation system comprising 79 lead rubber bearings and 49 flat sliders [52].

Compared to the new hospital constructions with seismic isolation, the number of hospitals retrofitted with seismic isolation are rather limited. Masuzawa and Hisada [37] investigated the retrofitting buildings by the seismic isolation in Japan. It is reported that, only 4% of the all isolated buildings belong to the retrofitted group and only 2% of the retrofitted buildings are hospitals. The important retrofit applications of base isolation retrofitted hospitals are follows.

Seismic isolation was selected as the retrofit methodology to provide functionality after exposure to large earthquakes for the Long Beach VA Hospital, a 12-story, about 40,000 m^2, concrete shear-wall structure, that was found to have high earthquake vulnerability. The 110 lead-rubber, 18 rubber and 26 sliding bearings were installed in 1995 at the base of all jacketed concrete basement columns, while the hospital remained in service during construction [49,9].

Two nine-story buildings of Hamamatsu Medical Center in Japan (a five-building health complex with 600 beds) were the first base isolation retrofitted hospital buildings in Japan [38]. For retrofit with seismic isolation, 89 seismic isolation units were placed in the basement columns while building functions and medical services continued.

In Turkey, under the “Public Private Partnership (PPP)” project of the Ministry of Health, 21 health projects (so called City Hospitals) were constructed (or under construction) with seismic isolation as of 2017. These are very large hospital complexes encompassing between 600 and 4000 beds and total floor areas reaching more than 1 million squares meters, at a total cost of about USD 23 billion.

This program has resulted as a rapid increase in the seismic isolation applications for hospitals throughout Turkey. Furthermore, a special project named “İstanbul Seismic Risk Mitigation and Emergency

Table 4
Comparison of ELF application limits used seismic isolation codes.

Code	ASCE/SEI 7–16	[11]	NTC-08	[51]
Limitation on Site Seismicity	No limit on S_1	–	–	S_1 at MCE < 0.6 g
Limitation on Site Class	A, B, C, D	1, 2	–	A, B, C, D
Max. Plan Dimension(m)	–	–	50	–
Max. Height of Building (m)	19.8	60	20	20
Max. Number of Stories	4	–	5	4
Limitations on eccentricity	–	3%	3%	5%
Period Range of Isolated Structure	$3T_{fixed} \leq T_{isol} \leq 5$ s at MCE_R	$T_{isol} \geq 2.5$ s	$3T_{fixed} \leq T_{isol} \leq 3$ s	$T_{isol} \leq 4$ s at MCE

Fig. 4. Picture of the hospital complex prior to earthquake retrofit.

Fig. 5. Plan of the building with representation of the different blocks (4-story blocks A1, A2, A3; 12-story blocks A4, A7, B4, B8; 13-story blocks A8, B7; 2-story blocks A5, A6; 3-story blocks B1, B2, B3, B5, B6).

Preparedness" (ISMEP), basically focused on seismic retrofit of public buildings in Istanbul, has undertaken the new construction of three major hospitals and the retrofit of one hospital complex with seismic isolation in Istanbul [23,24,26,27,29].

On the basis of the survey conducted by Engineering News Record (ENR) in 2017 (https://www.enr.com/articles/42366-the-10-largest-base-isolated-buildings-in-the-world), three of the largest 10 base isolated buildings (measured by total floor area) in the world are located in Turkey. Adana Integrated Health Campus, Adana, Turkey, with 430,000 m^2 floor space and 1512 isolator units was the second in line after the Apple Park building of 445,005 m^2 floor space, sitting on 700 isolator units in Cupertino, California. The fourth and the ninth ones are the: Isparta City Hospital, Isparta, Turkey (221,000 m^2, 903 isolator units) and the Erzurum Regional Research and Training Hospital, Erzurum, Turkey (180,000 m^2, 386 isolator units). One important hospital project under construction in Turkey is the Ikitelli Integrated Health Campus in Istanbul. The 790,000 m^2, 2,354-bed hospital complex sitting on 2050 isolator units, is expected to be the largest base-isolated building in the world when completed in late 2018.

5. Başıbüyük Training and Research Hospital, Istanbul, Turkey

The Basibüyük Training and Research Hospital complex, located in Istanbul, is retrofitted with the incorporation of a seismic isolation system in connection with the Istanbul Seismic Risk Mitigation and Emergency Preparedness Project (ISMEP-IPCU, https://www.ipkb.gov.tr/en/).

The hospital complex is composed of sixteen 2–13 story blocks with a total area of 113.000 m^2 and 750-bed capacity. Typical story height is 448 cm and the structure has three foundation levels that vary between − 13.59 and 0.0 m. A picture (prior to retrofit applications) and a layout plan of the hospital complex are given respectively in Figs. 4, 5. Fig. 6 shows the structural plan of the complex at ground floor level.

Marmara University Başıbüyük Research and Training Hospital was built in 1991 with a design based on the 1975 version of the Turkish Earthquake Design Code. Due to structural deficiencies found on the basis of the 1998 version of the code, the hospital structure was retrofitted in 2002, by adding reinforced concrete shear walls and

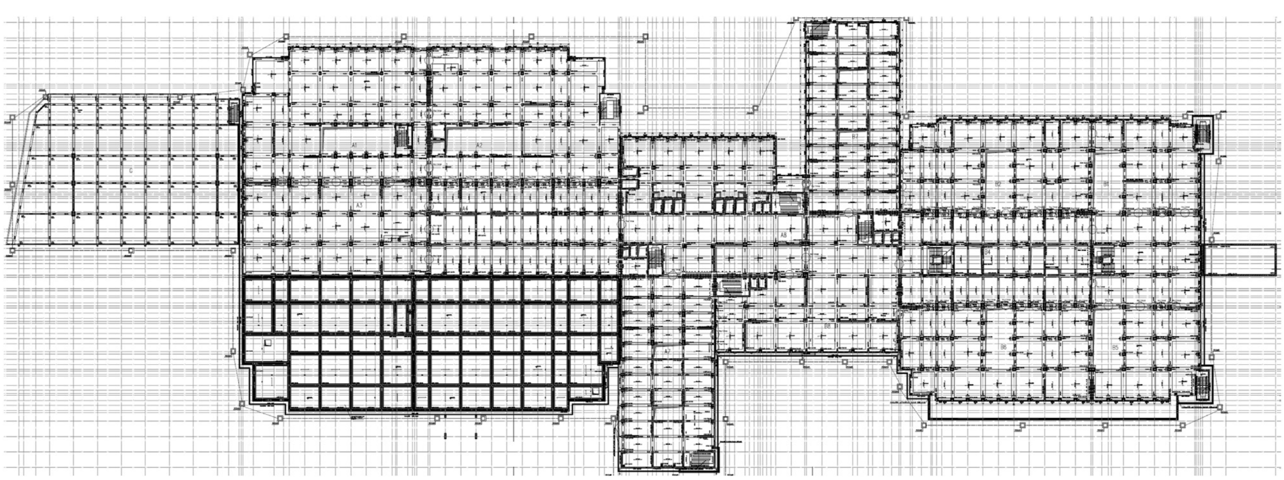


Fig. 6. Structural Section at Ground Motion Level.

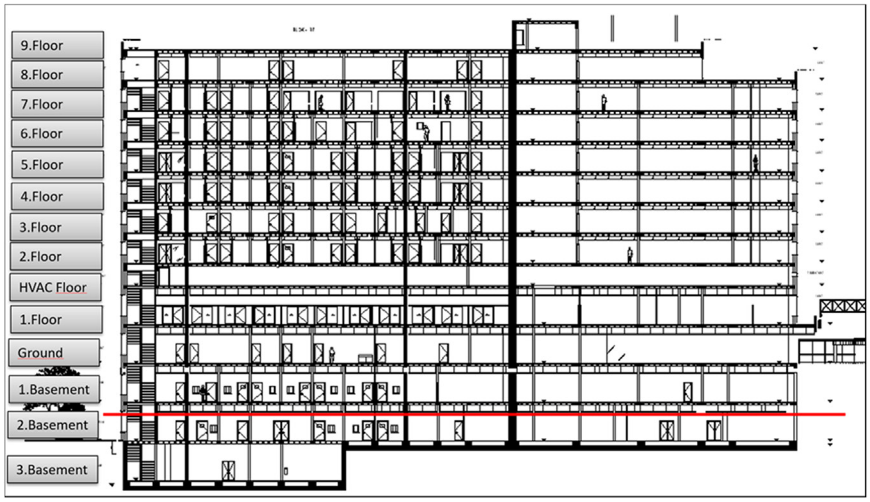


Fig. 7. Location of the isolation interface (shown with red line) on elevation section of one of the blocks. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.).

jacketing the columns. The hospital complex, which was not in service over this course of years, was structurally assessed again in 2011, this time on the basis of the 2007 version of code [48], which included a section on performance based assessment and retrofit design for existing buildings.

The structure was found to have inadequate capacity to satisfy the performance criteria required for hospitals (Life Safety and Immediate Occupancy performance levels for, respectively, the 2475-year and 475-year average return period earthquake ground motion levels) and earmarked for a second retrofit [35].

Although, conventional retrofitting methods (i.e. addition of shear walls and increasing the size of existing elements) increase the capacity of the structure to meet the demand, they also increase the floor acceleration (non-structural damage) and involve substantial architectural modifications (loss of functionality). The alternative retrofitting strategy is based on reduction of seismic demand through seismic isolation and energy dissipation. This strategy even decreases the floor acceleration and, for structures with appropriate capacity, does not modify the existing architecture (i.e. functionality is maintained), since most retrofit construction work is confined at the isolation interface.

Since the retrofit strategy had the performance objective of “immediate occupancy” and the functionality of the hospital is needed to be maintained, the seismic isolation method was selected to upgrade the seismic performance of the existing structure. Prota Inc., Ankara, has conducted the retrofit and renovation design with seismic isolation system in 2012. It is claimed to be the world's largest seismic retrofit project using seismic isolation.

For the design of the isolation system Chapter 17 of ASCE 7-10 [3] was considered. The design aimed to limit the base shear to 12.5% of the seismic weight of the superstructure at DBE (475-year) level and to keep the story-drifts under 0.5% and 1.0% respectively for DBE and MCE (2475-year) levels.

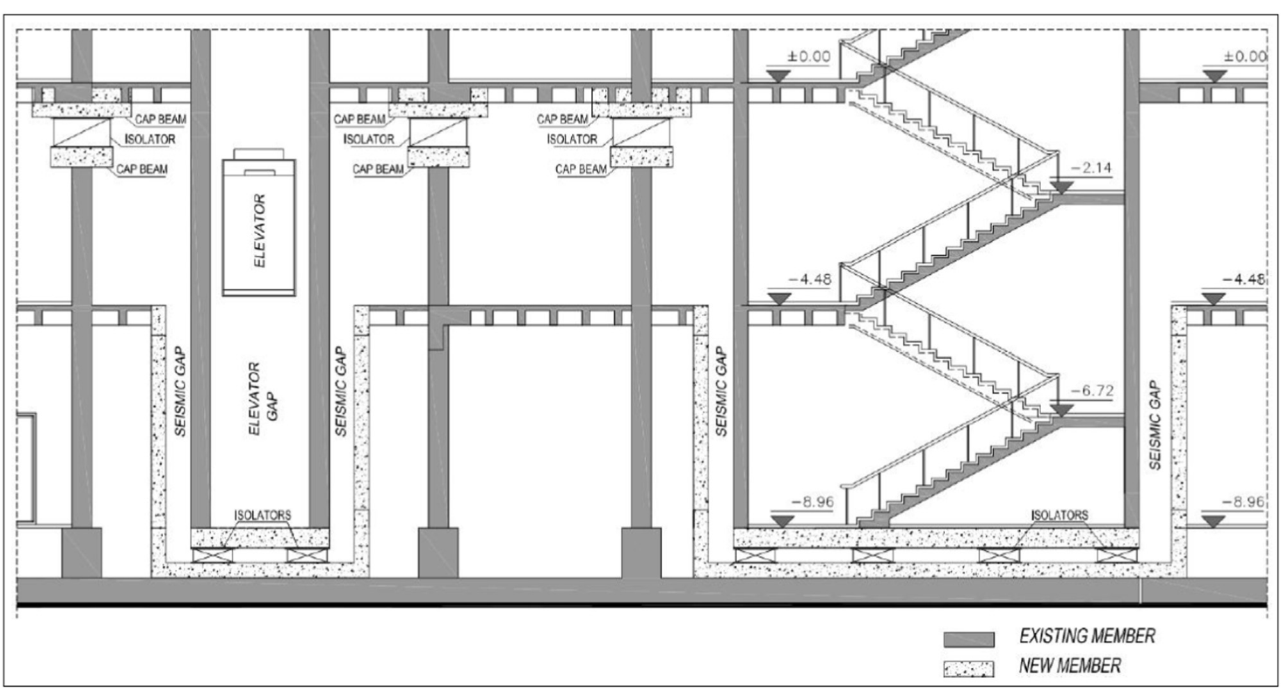


Fig. 8. Isolation interface location under the staircase and elevator core walls. Note the new structural elements (light grey) built to accommodate isolation at different levels.

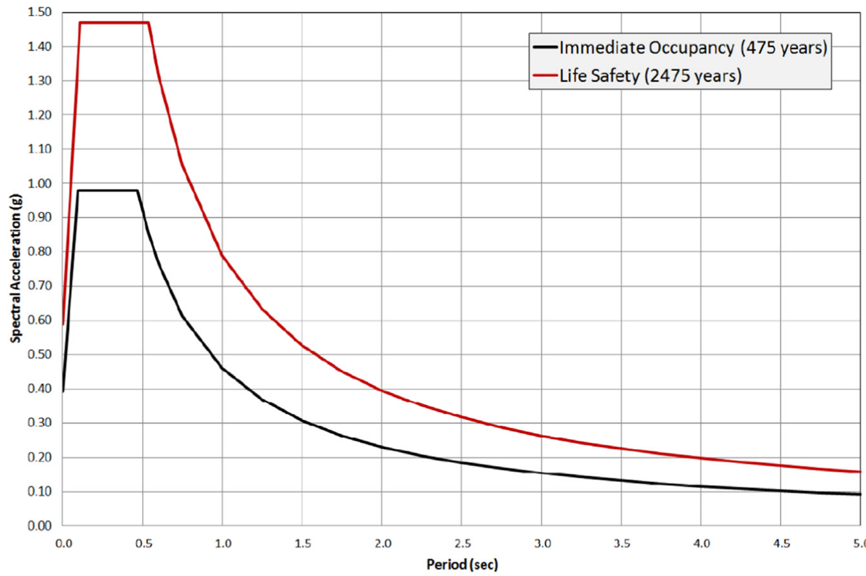


Fig. 9. Design basis acceleration spectra (5% damping ratio) for DBE (black) and MCE (red) ground motion levels. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.).

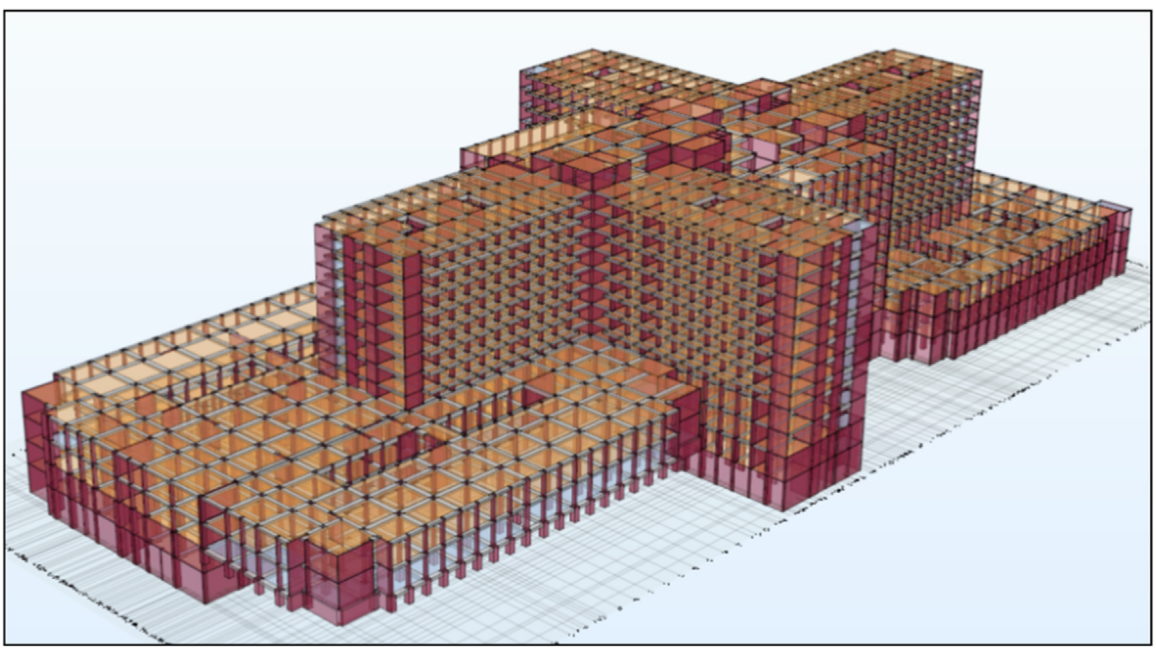


Fig. 10. Finite element model of the structural system.

Considering the architectural limitations and construction practice, the isolation layer has been selected as the top of the first basement floor, as indicated in Fig. 7. The columns below isolation layer were retrofitted through RC jacketing.

The stairs and elevators crossing the isolation interface constitute a problem in the retrofit design. To ensure the functionality of stairs and the elevators, the elevator and staircase core walls were supported with slider units at the foundation level as illustrated in Fig. 8. Needless to say, sufficient gap was implemented between the elevator/staircase core walls and the basement floor slabs to avoid collision.

Under the isolation interface, all blocks were merged into a single unit by jointly jacketing the adjacent columns and connecting the adjacent floors and shear walls. Similarly, peripheral shear walls were added and connected to the basement floors to provide the needed rigidity and strength in the substructure.

To eliminate risk of hammering and expensive seismic joint detailing, the floors of high-rise blocks were united at the super-structure level through special connectors. Analysis results proved that provided seismic gaps between low-rise blocks were sufficient, hence low-rise blocks were not united over the isolation interface.

The design basis response spectra for the 475- and 2475-year average return periods (i.e. DBE and MCE ground motion levels) were determined from the earthquake hazard map, associated with the earthquake resistant design code of railroad bridges and harbors of the Ministry of Transportation [18,32]. The design basis acceleration spectra are provided in Fig. 9.

A 3D finite element model of the building is constructed considering the nonlinear behavior of the isolation system with the force-displacement relationship provided by the manufacturer. Response spectrum analysis has been performed at DBE earthquake level and nonlinear

Table 5
Nominal MCE level physical properties of the LRB and NTM units.

Isolator Type	Quantity	D_{max} (mm)	F_{max} (kN)	K_{eff} (kN/mm)	F_y (kN)	K_1 (kN/mm)	K_2 (kN/mm)	ξ_{eff} (%)
LRB 650 × 283	129	380	435	1.15	147	7.96	0.80	28
NTM 300/43.5	146	380						
LRB 750 × 363	93	380	433	1.14	159	7.63	0.76	30
NTM 400/65	73	380						
LRB 800 × 297	97	380	630	1.66	223	11.29	1.13	29
NTM 500/102	76	380						
LRB 850 × 301	33	380	819	2.16	315	14.09	1.41	31
NTM 600/123.5	24	380						
LRB 1000 × 314	10	380	1185	3.12	503	19.27	1.93	33
NTM 700/127	6	380						
TOTAL	687	380	197592	520	71400	3509	351	30

response history analysis has been performed using the ground motion set for MCE level. The finite element model of the system is shown in Fig. 10. The structural model has been created and structural analysis has been conducted using Prota's in-house developed software, Probina Orion (www.probina.com.tr). All column, beam and shear wall elements were modelled as elastic frame elements while isolators were idealized by elastic link elements.

Generally elastomeric and curved surface friction type bearings are used for the retrofit of building structures with seismic isolation. For cases where elastomeric bearings are selected, different diameter isolator units are used in consideration of different levels of vertical loads and to ensure their stability under lateral displacements. The increase in bearing diameters results in stiffer bearings and creates difficulty in effective isolation of structures with light column loads. As such the use flat sliding bearings in the isolation system becomes necessary to reduce the lateral stiffness (and consequently the isolation frequency) of the isolation system.

The isolation system that will be used for the retrofit was determined as a combination (687 units) of lead rubber (LRB) and sliding (NTM) bearings manufactured by Freyssinet Inc. in conformity with the EN 15129 [22] regulations. Furthermore, 154 Pot type supports were installed under the stairwells and elevator cores (Fig. 8). The nominal physical properties of the five types of LRB and NTM units are provided in Table 5. In this table: D_{max} = MCE level maximum displacement, F_{max} = Lateral force at D_{max}, F_y = Yield Force, K_{eff} = Effective stiffness at MCE level, K_1 = Elastic Stiffness, K_2 = Post-elastic Stiffness and ξ_{eff} = Effective damping ratio at MCE level.

Prototype (Characterization) tests of LRB bearings have been performed at EUCENTRE Laboratory in Pavia based on the testing criteria defined in EN 15129 [22]. The MCE level of resistance and maximum displacement reported in Table 5 (ELF analysis results) were considered for testing. Force-displacement results for LRB 800 × 297 under horizontal cyclic and lateral capacity tests are provided in Fig. 11. A summary of test results for DBE Level parameters are provided in Table 6.

NTM sliding supports, shown in Fig. 12, consist of a series of rubber layers with steel shims and a steel sliding plate. The sliding surface consists of PTFE coupled to a sheet of stainless steel.

The installation of the LRB units together with specially designed NTM sliding supports provided the vertical support in all of the columns without increasing the lateral stiffness of the isolation system. Special studies were conducted to assess the effect of differential settlement of LRB, NTM and Pot bearing units on the structural system. The differential deflection induced by the horizontal displacement in seismic condition was always kept below 2 mm for each family of devices.

For NTM flat slider bearing units, vertical load and friction coefficient tests were conducted. During the vertical load test, the bearing is vertically loaded at the vertical load level given at Table 7 and vertical deflections are measured. Similarly, during the friction tests, the bearing is loaded with the corresponding vertical load level given at Table 7 and displaced 380 mm horizontally while maximum horizontal force is recorded. The dynamic friction coefficient values obtained from the friction tests were found to be lower than 0.01. Table 7 provides the vertical load values used in the tests.

Below the stairwell and elevator cores, sliding supports (Pot bearings) were installed to support their weight and, at the same time, allow their movements due to the earthquake. These Pot bearings consist of confined elastomeric disc bearings fitted with a steel and PTFE sliding plate (ALGAPOT PNm – Free Sliding Bearings). Pot bearings are conceptually similar to the NTM units (Fig. 13), however with no rubber

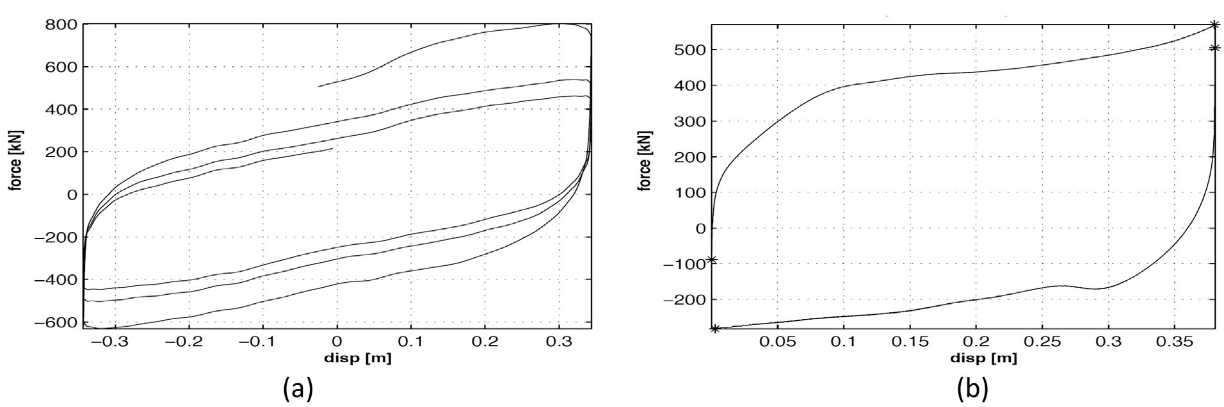


Fig. 11. Force–displacement relationship of LRB 800 × 297 under 4MN axial load. (a) Horizontal Cyclic Characteristics (real time, 3 cycles Maximum Displacement = 342 mm); (b) Lateral Capacity (Ramp test, Maximum Displacement = 380 mm).

Table 6
Prototype Test Results at DBE Level (Mean of 3 cycles and Range shown in parenthesis).

	LRB 650 × 283	LRB 750 × 363	LRB 800 × 297	LRB 850 × 301	LRB 1000 × 314
K_{eff}	1215	1204	1763	2305	3356
(N/mm)	(972 – 1458)	(954 – 1445)	(1410–2116)	(1844–2766)	(2685 – 4027)
ξ_{eff}	30.7	32.1	31.5	33.0	34.9
(%)	(24.5 – 36.8)	(25.7 – 38.5	(25.2 – 37.8)	(26.4 – 39.6)	(27.9 – 41.8)

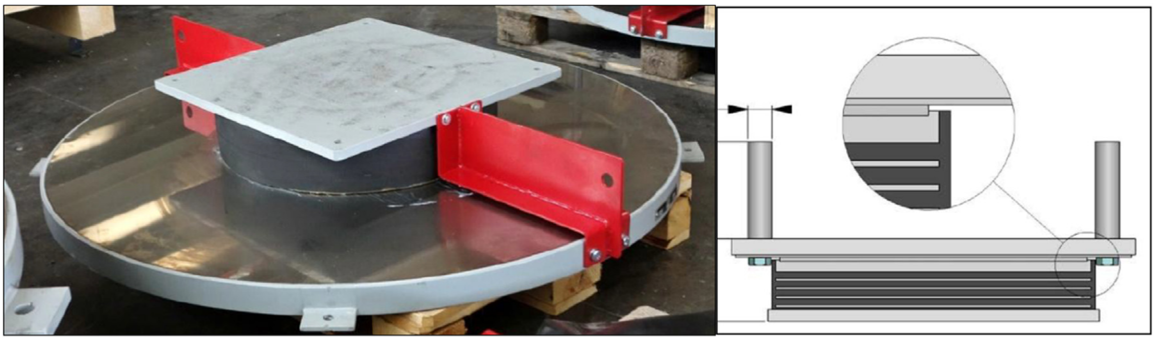


Fig. 12. NTM sliding support with temporary fixing plates (red). (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.).

Table 7
NTM Flat Slider Testing Protocol.

Bearing type	Vertical load test V (static) [kN]	Friction test V (seismic) [kN]
NTM 300 × 43.5	2017	1000
NTM 400 × 65	3925	2000
NTM 500 × 102	7470	4000
NTM 600 × 123.5	11220	6000
NTM 700 × 127	15564	13068

Table 8
POT Bearings Testing Protocol.

Bearing type	Vertical load test V (static) [kN]	Friction test V (seismic) [kN
PNM 2218 / 810 / 810	2018	247–2218
PNM 5627 / 810 / 810	5627	1811–5627
PNM 8064 / 810 / 810	7841	4420–8064

layers and steel shims. Table 8 provides the vertical load values used in the tests of POT bearings very similar to the test procedure of NTM devices described previously.

Fig. 14 shows the plan layout of the isolation system. The distribution of the isolation units was determined to minimize the torsional response of the superstructure. The horizontal distance between the center of stiffness of the isolation system and the projection of center of mass of the superstructure on the isolation interface, was kept as low as 1.5 m.

The Equivalent Lateral Load (ELF) analysis results used for the seismic isolation design are provided in Table 9 (After Prota Inc.).

The design forces and displacements were further obtained by nonlinear response history analysis of the 3D finite element model of the structure and using 7 sets of spectrum compatible accelerometric data.

The high-rise blocks were modelled as a single structure above the isolation interface, since they were united by special connectors. On the other hand, low-rise blocks were modelled individually, since they were separated by expansion joints and the clear gap between these blocks were proved to be sufficient by analysis results. In the finite element model all column, beam and shear wall elements were modelled as elastic frame elements while isolators were modelled by idealized link elements.

Results of base shear and maximum isolator displacement at the

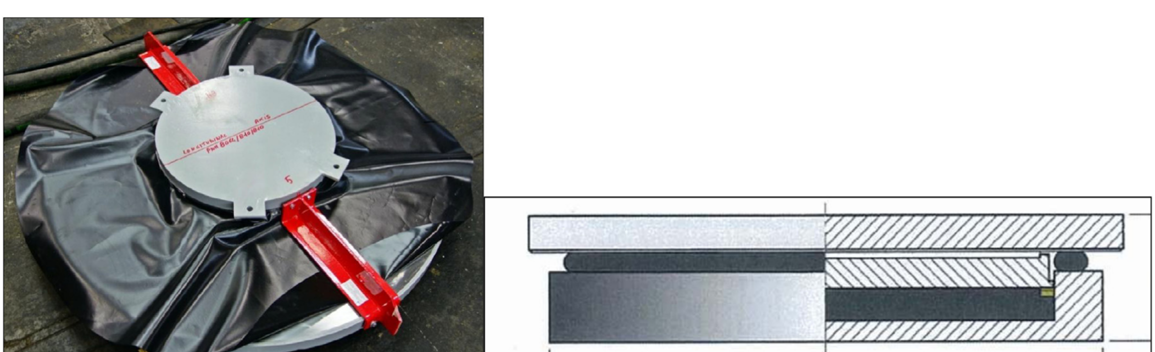


Fig. 13. Pot type flat slider bearings (installed under the stairwells and elevator cores).

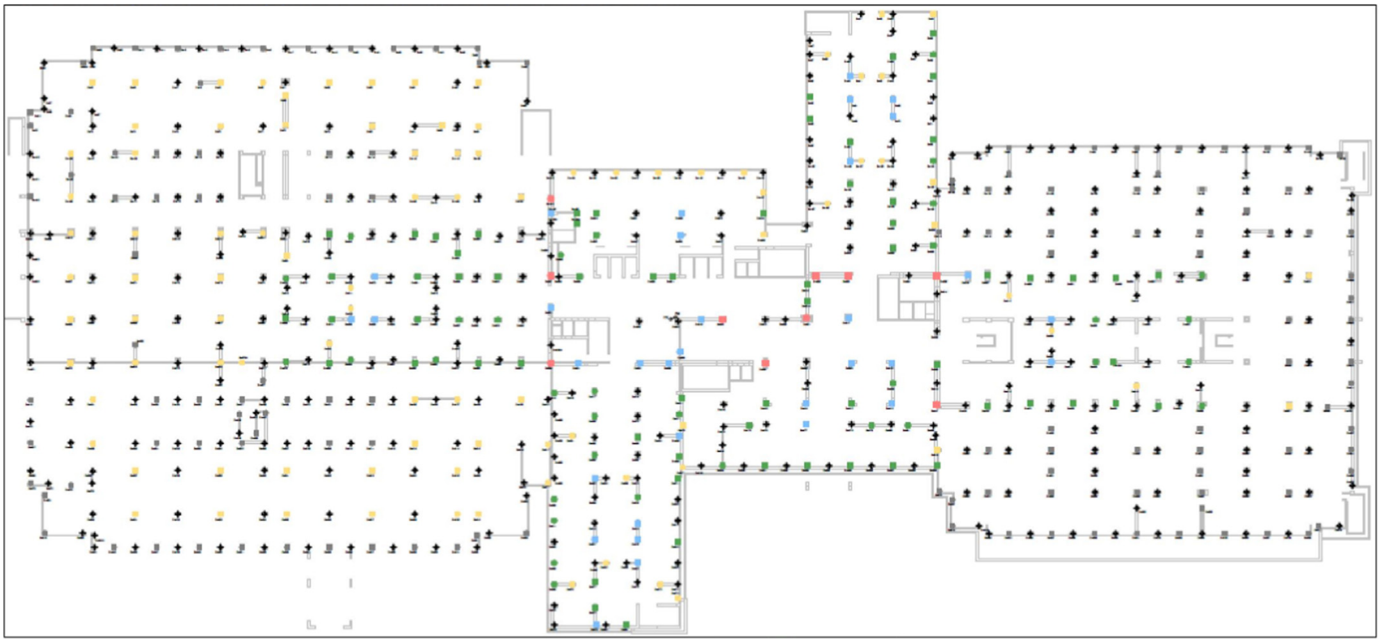


Fig. 14. Layout of the Isolator Types in the Isolation Interface (Black = NTM units, Grey = LRB 650 ×283, Yellow = LRB 750 × 363, Green = LRB 800 × 297, Light Blue = LRB 850 × 301, Pink = LRB 1000 ×314). (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.).

Table 9
Equivalent Lateral Force (ELF) analysis results.

Total Dead Load	1.20 GN
Total Live Load	0.55GN
Seismic Weight	1.39 GN
DBE Level Isolated Period	2.63 s
MCE Level Isolated Period	3.29 s
DBE Level Isolation System Center of Resistance Displacement	140 mm
MCE Level Isolation System Center of Resistance Displacement	339 mm
MCE Level Isolation System Maximum Displacement	373 mm
DBE Level Effective Damping Ratio	41%
MCE Level Effective Damping Ratio	32%
DBE Level Lateral Load Coefficient	8.4%
MCE Level Lateral Load Coefficient	12.7%

MCE level are provided in Table 10 (After Prota Inc.)

The time-domain nonlinear analysis results indicate the verification of the MCE (Life Safety) and DBE (Immediate Occupancy) level performance criteria at each independent block. Story drifts in orthogonal directions (x and y) for the high-story blocks (blocks A4, A7, A8, B4, B7 and B8 in Fig. 5) under DBE and MCE level earthquake ground motion levels are illustrated in Fig. 15.

The technology used for the installation of isolation units constitute a fundamental part of the retrofit design with seismic isolation. It concerns the method of the temporary transfer of the vertical loads during the cutting operation of the columns, the installation of the isolator and the possible future replacement operations. Furthermore, special measures are needed for stairs, elevators and lifelines crossing the isolation level.

Table 10
MCE Level Base Shear and Maximum Isolator Displacement (After Prota Inc.).

Scaled Accelerometric Data Obtained From	Base Shear (MN)	Max Disp. (mm)
Parkfield, California (C12)	149	293
Imperial Valley, California	106	93
Düzce, Turkey (1061)	164	245
Landers, California	163	302
Düzce, Turkey (362)	122	152
Morgan Hill, California (CLS)	117	138
Parkfield, California (TMB)	159	239
Average	140	209

Briseghella [10] provides a treatment of different retrofitting strategies of existing buildings, including “column cut” techniques and “Lift up” systems. Castellano [14] provides a survey of Italian experience in seismic retrofit of buildings through seismic isolation and reports on the intervention technology and the methodology for the installation of the isolators.

Before the installation process, the designer provided an installation technology and sequence in order to avoid any vertical deflection that may cause damage in the structural members above isolation layer. The technology consisted of the application of two special steel clamps, to each column, placed with a suitable distance between them (about 40–50 cm). These clamps are then tightened to the column by means of prestressing special bars of high strength steel. Subsequently, by means of suitable hydraulic jacks, the column section included between the two clamps is unloaded. By means of cuts with a diamond wire saw, a section of suitable height of the unloaded column section is removed and the isolation unit is inserted in its place with appropriate grouting and anchoring to the cut surfaces of the column. Once the grout is set, the jacks are unloaded and the vertical force in the column is transferred to the isolator unit. The installation process is shown in Figs. 16, 17.

During cutting and installation process vertical deflection is monitored in order to avoid any loss of post tension force and to maintain the original height of the column. The isolator unit installation process was carried out by Freysaş Inc., sub-contracted by the general contractor ZEK-SAN + BEGÜM YAPI and under the supervison of STIPE + timA + Khatib Alami. The first author of this paper served as technical supervisor. A picture of the retrofitted Maltepe Basibuyuk Training and Research Hospital complex is provided in Fig. 18.

6. Adana Integrated Health Campus, Adana, Turkey

Adana Integrated Health Campus, Adana, Turkey, is developed as a public-private partnership between ADN PPP Sağlık Yatırım A.Ş. (a

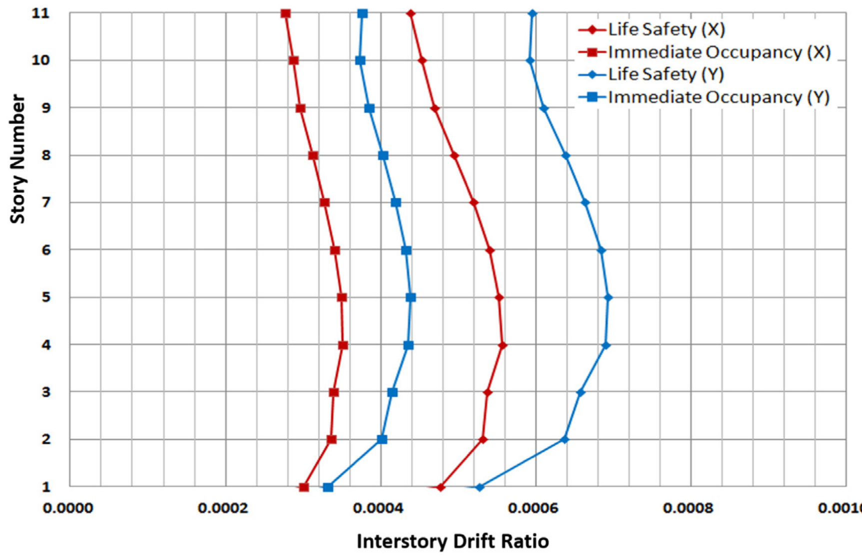


Fig. 15. Intertory-drift ratio profile for DBE (red) and MCE (blue) levels in two orthogonal directions. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.).

joint venture of four firms) and the Turkish Ministry of Health. The campus has a total capacity of 1550 beds housed in three hospital units: the 1,300-bed main hospital, a 150-bed physical-therapy and rehabilitation hospital and a 100-bed high-security criminal /psychiatric hospital. The campus building is supported by 1512 base isolators. The architectural design of the complex was done by HWP Planungsgesellschaft mbH, Stuttgart, Germany. The structural engineer was Ülker Engineering Ltd., Istanbul, Turkey. It was built by Rönesans Sağlık Yatırım Inc., Ankara, Turkey and was ready for medical services in May 2017. The first author of this paper served as peer reviewer.

Adana Integrated Health Campus, encompassing 430,000 m^2 floor space, was identified as the second largest base isolated building (measured by total floor area) in the world (after Apple Park, building of 445,000 m^2, sitting on 700 isolator units in Cupertino, California) on the basis of the "The 10 Largest Base-Isolated Buildings in the World" study in 2017 conducted by Engineering News Record (ENR) (https://www.enr.com/articles/42366-the-10-largest-base-isolated-buildings-in-the-world).

The main building within the Adana Integrated Health Campus Complex houses the general hospital, oncology hospital, obstetrics hospital, cardiovascular hospital and the psychiatry hospital with a total floor area of 430.000 m^2 and a total bed capacity of 1.300. The hospital structure, located on a high seismic zone, is supported by 1513 triple friction pendulum seismic isolators and targets "operational" performance level under the Design Basis Earthquake (DBE). The complex is located on an octagonal base (Fig. 32), where 4 towers with 11 stories stand at each corner in a symmetric fashion. A render view is shown in Fig. 19.

The foundation system consists of a reinforced concrete mat below the tower blocks and two-way grade beam below the low-rise blocks. The thickness of the mat foundation varies between 160–200 cm and the grade beams are 200 cm (width) by 80 cm (depth). With the exception of core wall isolators, seismic isolators are located atop the first floor (second basement) columns. The second-floor slab (first basement floor) is a single reinforced concrete diaphragm with no joints with a total thickness of 60 cm. After the second floor slab, the structure is separated into 17 individual blocks. A layout of the building blocks located in the hospital complex is shown in Fig. 20. Figs. 21, 22 provide, respectively, the structural plan at the ground floor (above the isolation interface) and a structural elevation cross-section. The location of the

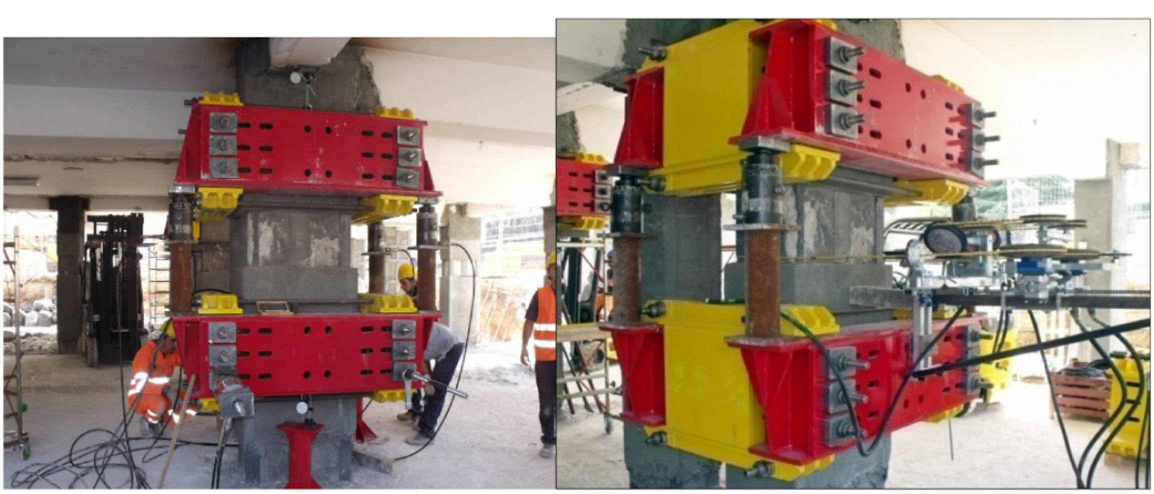


Fig. 16. Jacking, diamond wire cutting and extraction of the column block.

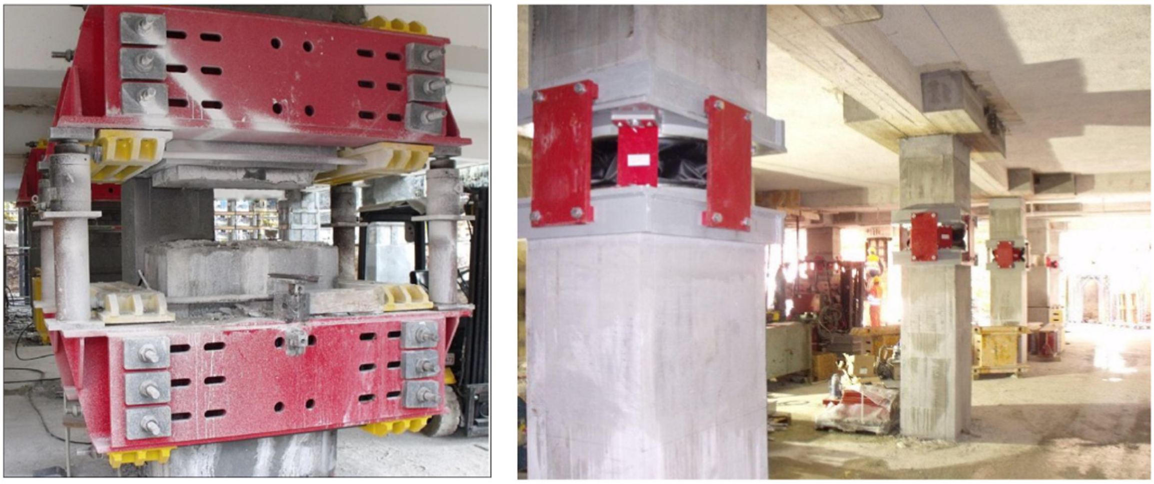
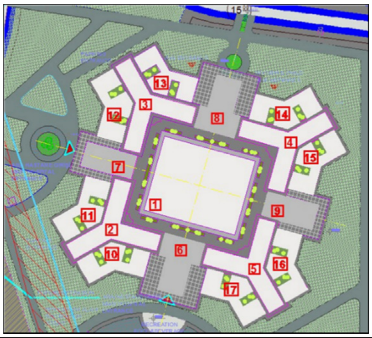


Fig. 17. Installation of the isolation units (locked) at the top of first basement columns.

Fig. 18. Maltepe Basibuyuk Training and Research Hospital after retrofit.

Fig. 19. 3D rendering of Adana health complex.



Block No	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Stories	8	13	13	13	14	7	5	5	5	5	5	5	5	5	5	5	5

Fig. 20. Block Configuration.

structural elevation cross-section is indicated in Fig. 21. The story heights and the story elevations for the main block are indicated in Table 11.

Lacking a national official code at the time of the design, ASCE 7–10 [3] "Minimum Design Loads for Buildings and Other Structures" Chapter 17 "Seismic Design Requirements for Seismically Isolated Structures" was used, in general, during the design process. The structure concrete classes in accordance with EN 206 (2000) are C50/60 for pedestals supporting the seismic isolators and C40/50 for the remaining elements. The structural steel grade is B420C (Yield stress = 420 MPa; Ultimate stress = 550 MPa) as per TS 708 (Turkish code for reinforcement steel).

In order to determine the seismic hazard at the site and the design basis ground motion a probabilistic seismic hazard study was conducted [28]. General soil profile for the site is characterized by silty sandy clay with an allowable bearing pressure of 400kPa and a vertical subgrade modulus of 50.000 kN/m^3. The shear wave velocity of the site varies between Vs30 = 400–700 m/s with SPT-N values in the range of 30–50. Accordingly, the site was classified as a NEHRP-C site class. The site specific spectral acceleration parameters are provided in Table 12 and the Design Basis Earthquake (DBE) and the Maximum Considered Earthquake (MCE) level ground motion spectra are plotted in Fig. 23. The DBE is defined as a seismic event with a 10% probability of exceedance in a 50 year interval (equivalent to an earthquake with a 475 year return period). The MCE is defined as a seismic event with a 2% probability of exceedance in a 50 year interval (equivalent to an earthquake with a 2475 year return period.)

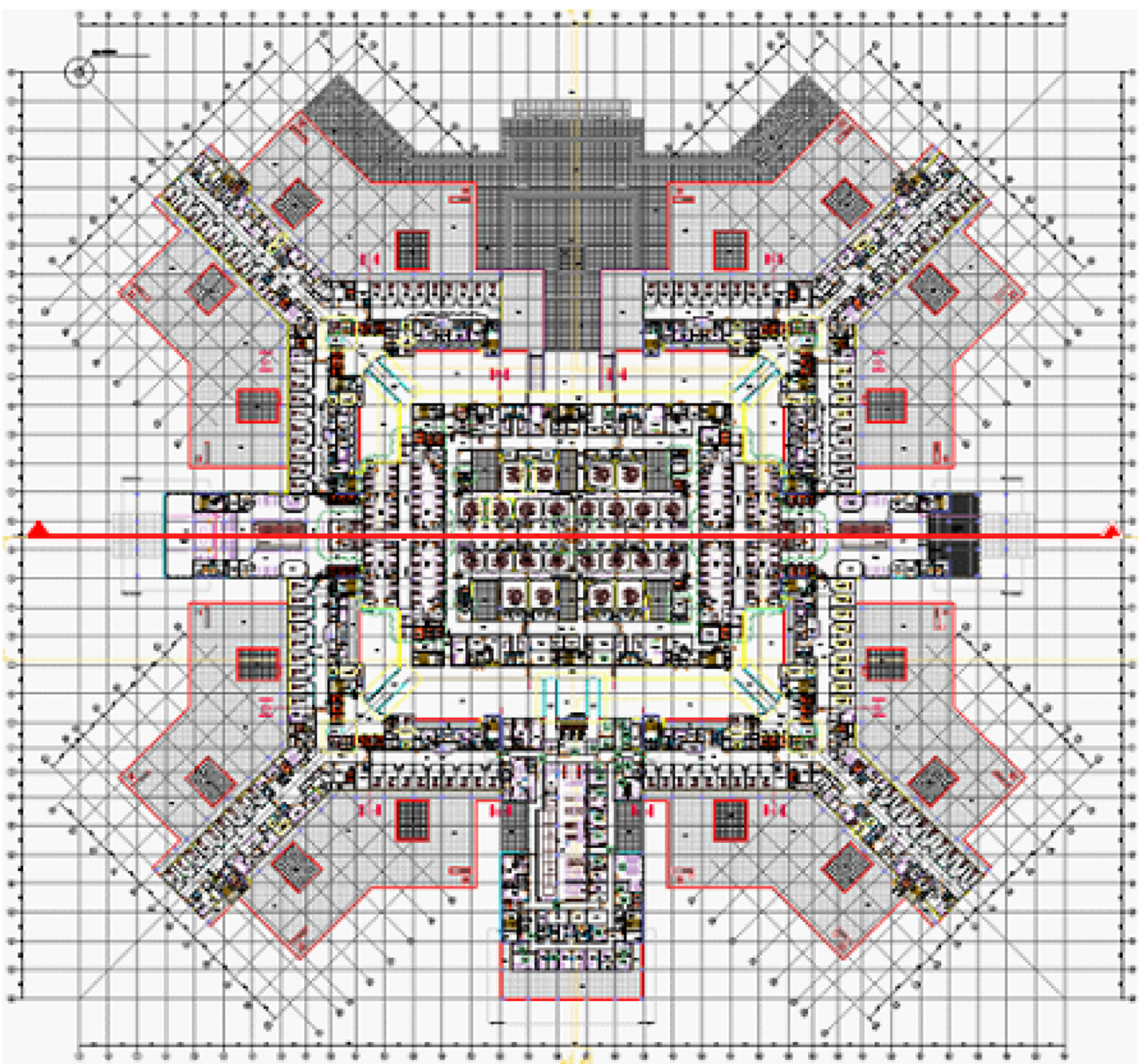


Fig. 21. Structural plan at the ground floor (above the isolation interface).

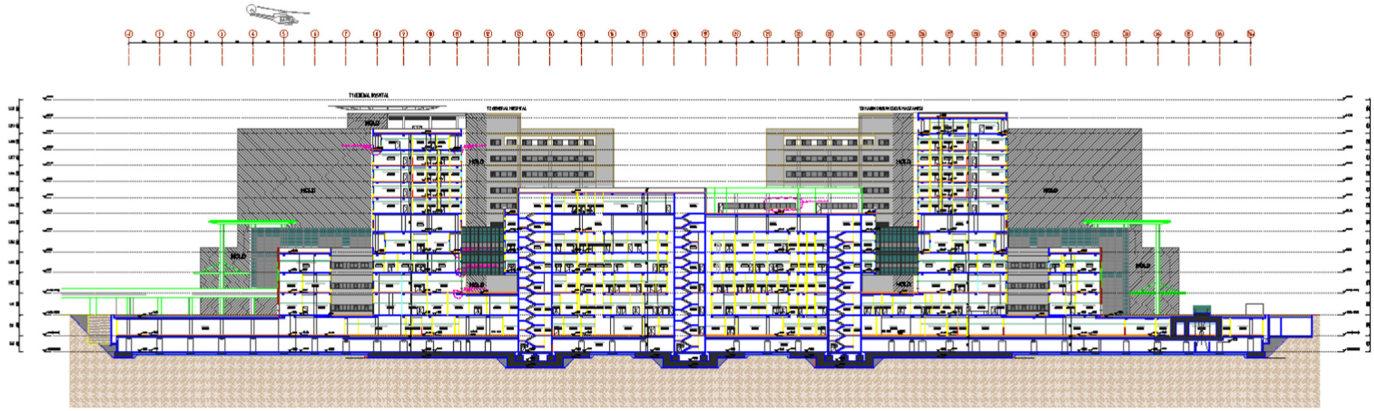


Fig. 22. Structural elevation cross-section (section indicated with red line in Fig. 21).

Table 11
Story heights and elevations.

STORY	HEIGHT (m)	ELEVATION (m)
Roof	4.3	52.30
9th Floor	5	48.00
8th Floor	4.3	43.00
7th Floor	4.3	38.70
6th Floor	4.3	34.40
5th Floor	4.3	30.10
4th Floor	4.3	25.80
3rd Floor	5	21.50
2nd Floor	5.5	16.50
1st Floor	5.5	11.00
High Entrance	5.5	5.50
Low Entrance	6	0.00
1st Basement	5.5	− 6.00
2nd Basement	4.5	− 11.50
Foundation		− 16.00

Table 12
Site-Specific Spectral Parameters (NEHRP Site Class C).

Return Period (years)	Level	Ss (0.2 s)	S1 (1.0 s)
475 years	DBE	0.78 g	0.29 g
2475 years	MCE	1.30 g	0.53 g

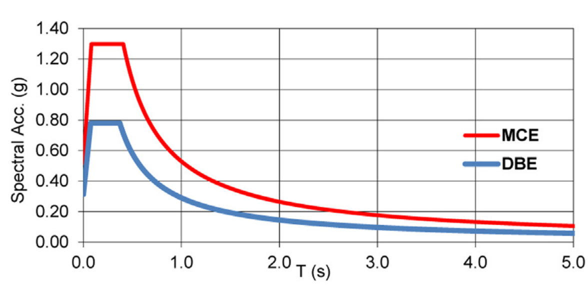


Fig. 23. Site specific design basis acceleration spectra for DBE and MCE level.

Based on the determined spectral parameters, seven pairs of spectrum compatible ground motion acceleration time histories were obtained for both return periods, considering ASCE 7–10 [3] criteria. It should be noted that the vertical acceleration records were not included in the dynamic analysis; however they were taken into account using

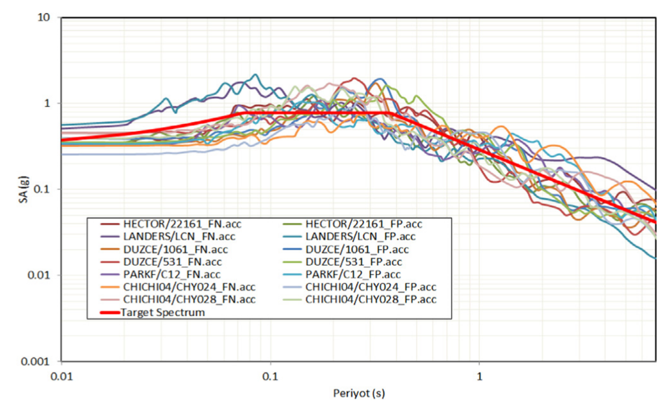


Fig. 24. Comparison of acceleration spectra of scaled ground motion with the target spectrum at DBE level ground motion.

ASCE 7–10 load combinations. Figs. 24, 25 provide comparisons of the spectra of scaled accelerograms with the target spectra for DBE and MCE ground motıon levels.

The Seismic Performance Objectives for the Adana Integrated Health Campus was set as “Operational” or “Continued Functionality” under exposure to the DBE level (10% probability of exceedance in 50 years) and “Immediate Occupancy” under exposure to the MCE level (2% probability of exceedance in 50 years) ground motion. To meet the operational performance objective at the DBE level ground motion: the inter-story drift ratio will be under 0.005 and the peak floor accelerations will stay under 0.2 g level, such that damage to the non-structural elements and the equipment is avoided. The inter-story drift ratio under exposure to MCE level ground motion will be less than 0.01 to ensure immediate occupancy performance objective. For preliminary analysis, using a linear analysis procedure, the Response Modification Coefficients of 1.0 and 1.5 shall be assumed respectively for the DBE and MCE level design for the superstructure to meet the performance objectives in the structural sense.

At the beginning of the design, the isolation system was selected to be composed of curved surface slider type isolation units. The primary reason for this selection was the bearing supply being on the critical path during construction. The secondary reasons were the avoidance of torsion and tension problems, since for curved surface sliders the center of resistance of the isolation system matches the center of mass of the superstructure and the isolation units can separate (and then unite again) if uplift occurs. An “uplift test” is conducted in the prototype testing sequence to verify the integrity and safety of the isolator units.

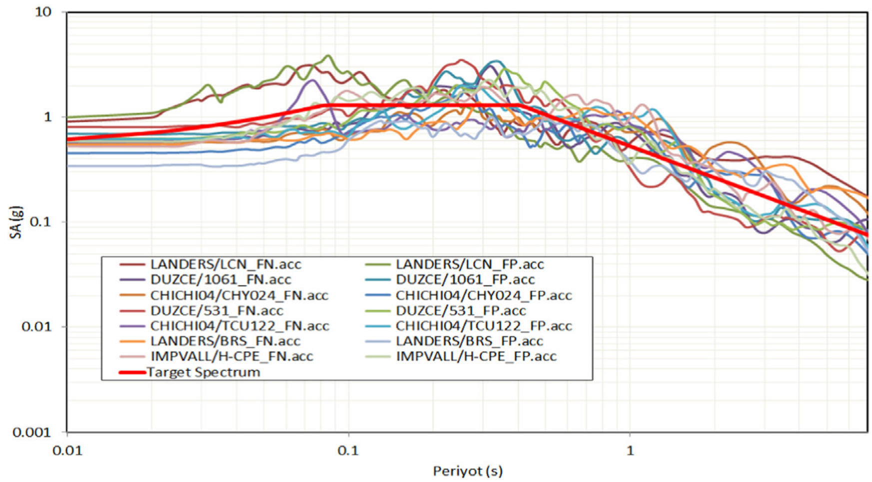


Fig. 25. Comparison of acceleration spectra of scaled ground motion with the target spectrum at MCE level ground motion.

The requirements for the isolator units (in addition to those stipulated in the design codes) were set as follows:

(1) An upper bound and lower bound analysis shall be executed. Unless testing data is provided 0.8 μ_{nom} and 1.69 μ_{nom} (μ_{nom} stands for the nominal friction coefficient) shall be assumed for lower and upper bound coefficient of friction respectively.
(2) Sliding units shall be able to carry safely an axial load of 1.5 (DL + SDL + LL).
(3) Sliding units shall be able to resist an axial load of 0.8DL – |E| and 1.2(DL + SDL) + LL + |E|, considering the MCE level ground motion.
(4) The maximum variability of the coefficient of friction considering single isolator units shall be less than 15% and maximum variability of the coefficient of friction considering entire system shall be less than 5%.

Where, DL, SDL, LL and E denotes respectively the Dead load, Superimposed dead load, Live load and Earthquake load.

A three-tier analysis and design methodology was used. The procedure followed during analysis and design can be summarized as follows:

Stage – I

For preliminary design of the isolation system, a base shear of 10% of seismic weight is targeted for DBE level using upper bound isolator properties and a maximum displacement of 40 cm is targeted for MCE event using lower bound isolator properties. Regardless of the earthquake level, a maximum of 30% damping shall be indicative at the isolation level.

In connection with this preliminary design, the column (isolator) axial loads and displacements at the isolation interface are computed. The design was targeted for ordinary moment frame and shear wall system based on the ACI 318-11 [4] code and compliance with Turkish Seismic Code (2007) [48] was ensured.

These loads and displacements were then provided to the seismic isolator manufacturers to submit their isolation system proposals with relevant parameters including (a) coefficient of friction versus axial pressure plots for all surfaces; (b) radius of curvature for all surfaces; (b) upper and lower bound coefficients of friction considering aging, environment, testing and production.

Stage – II

Upon submission of the isolator parameters, the model was developed and detailed to include the isolator nonlinearity and the sub structure. Based on the data obtained from the manufacturer related to the isolator properties, 7 nonlinear direct integration response history analyses were performed in two orthogonal directions for the two different ground motion levels. The super structure was modelled as linear elastic. For the DBE analysis upper bound isolator properties were used to capture the maximum shear force, for the MCE analysis lower bound isolator properties were considered to obtain the maximum isolation system displacement and deflections.

At this stage, multiple iterations were executed to determine the optimum isolator distribution and properties. ASCE 7–10 [3] code minima, based on equivalent lateral force (ELF) analysis (Table 14), were checked for compliancy. At the end of this stage isolation system was finalized and the isolator test protocols were prepared.

Stage – III

Following the prototype tests, final modifications were done for the isolator properties and all NTHA were run for again for design verification. Based on the average of the results of non-linear response history analysis obtained from seven sets of accelerometric data, the structural design was finalized.

Based on the results of the verification analysis, detailed design of the structure was realized. ACI 209 [2] was used to determine in plane behavior and creep and shrinkage of the podium diaphragm above the isolation level. Furthermore, P-Δ effects were considered for the design of the sub structure.

ETABS 2013 (CSI, www.csiamerica.com) software was used for the analysis and the design of structural framing and SAFE V12 (CSI, www.csiamerica.com) software was used for the design of the slabs and the foundation elements. Fig. 26 illustrates the finite element model of the structure. The structure above the isolation system has a total seismic mass of 0.79 10^9 kg, which includes 30% of the live loads.

Based on the axial load values, three different seismic isolator types (triple curved surface slider units or triple friction pendula) were considered for the isolation system. The isolator units were produced by and supplied by Earthquake Protection Systems (www.earthquakeprotection.com). Fig. 27 shows the geometry of a triple friction pendulum bearing and its parameters (Fenz and Constantinou [33]). From top to bottom, the bearing consists of: top concave plate, top slide plate, rigid slider, bottom slide plate and bottom concave plate. Its behavior is characterized by R_i, the radius of curvature of surface i, h_i, the radial distance between the pivot point and surface i and μ_i, the coefficient of friction at the sliding interfaces. Typically, $h_1 = h_4$ and $h_2 = h_3$; $d_1 = d_4$ and $d_2 = d_3$ and; $\mu_1 = \mu_4$ and $\mu_2 = \mu_3$. The effective radius of curvature, $R_{eff,i}$, is equal to $R_i - h_i$.

Nominal values of the physical parameters of the three isolator

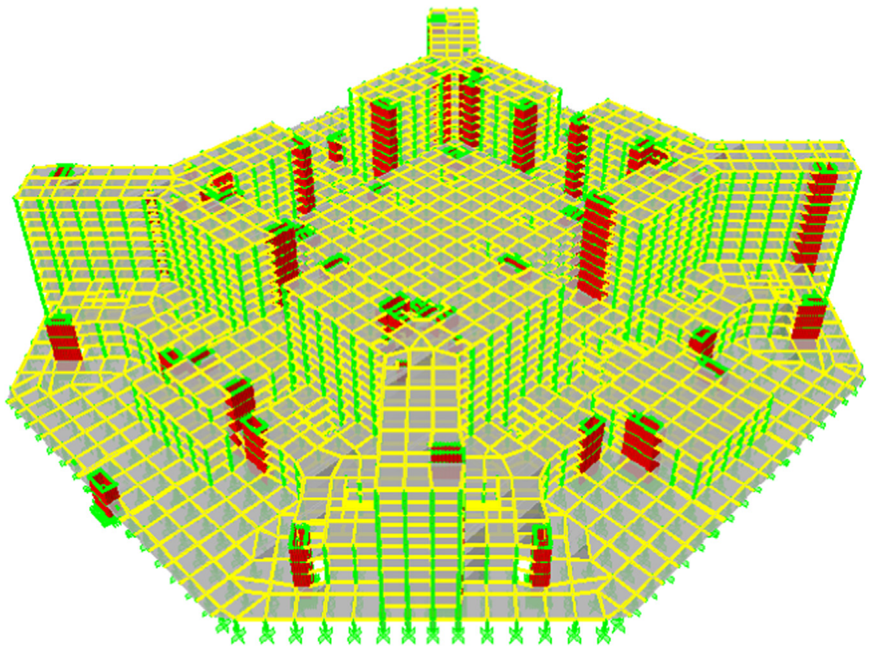


Fig. 26. Isometric view of the structural model.

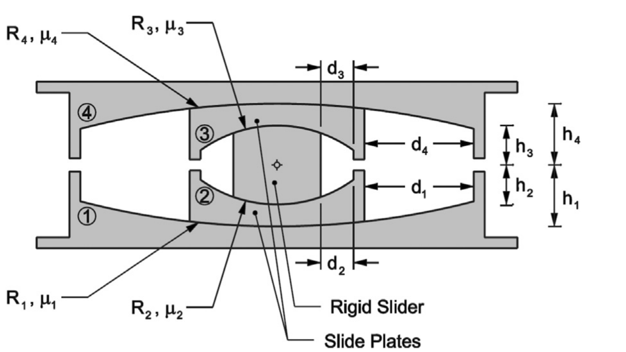


Fig. 27. Cross section of the triple friction pendulum bearing labeled with parameters that characterize its behavior.

types used in the isolation system are provided in Table 13. P_{el} and P_{ul} stand for the elastic and ultimate axial load capacity of the isolator, Δ_{mc} is the allowable maximum displacement capacity and, Δ_{tmc} is the absolute maximum displacement capacity of the seismic isolator.

The upper and lower bound friction coefficients are provided in Table 14 for each isolator type. Subscripts "lb" and "ub" respectively denote the lower and upper values of the coefficients of friction.

The cross section and nominal force- displacement relationship for one of the isolator types FPT 8836/22-12/10-8 are shown in Fig. 28 and Fig. 29, respectively.

The isolator units have been tested in EPS Inc. facilities, based on the procedure defined in Chapter 17 of ASCE 7–10 [3] code. The tests have been performed dynamically with the actual response velocities.

Table 14
Isolator upper and lower bound properties.

Type	$\mu_{1,lb}$	$\mu_{2,lb}$	$\mu_{1,ub}$	$\mu_{2,ub}$
FPT8836/22–20/16–8	0.007	0.03	0.015	0.05
FPT8831/18–16/12–7	0.007	0.03	0.015	0.05
FPT8827/14–12/10–6	0.007	0.03	0.015	0.05

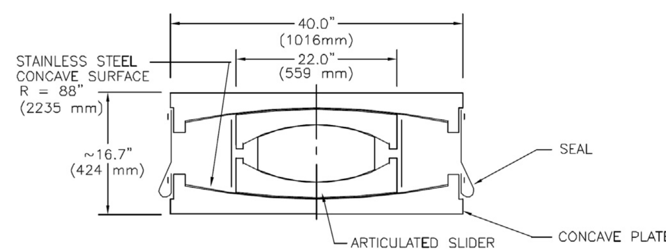


Fig. 28. Cross section of FPT-8836/22-20/16–8 triple pendulum isolator (after EPS Inc.).

The results of the prototype tests have been evaluated and parameters in terms of friction coefficient, effective stiffness and effective damping have been checked with the acceptance criteria defined in ASCE 7–10 [3]. A sample test result (Seismic properties test, ASCE 7–10 17.8.2.2.2) that indicates the three cycle force-displacement loop is provided in Fig. 30. Additional tests were also conducted to test the reserve capacity of the isolator units for energy dissipation, uplift, vertical load capacity

Table 13
Isolator parameters.

Type	Quan-tity	R_1 (mm)	R_2 (mm)	μ_1	μ_2	P_{el} (MN)	P_{ul} (MN)	Δ_{mc} (mm)	Δ_{tmc} (mm)
FPT8836/ 22–20/16–8	97	812	4165	0.01	0.04	35.9	53.8	368	406
FPT8831/ 18–16/12–7	297	635	4165	0.01	0.04	20.6	30.3	335	406
FPT8827/ 14–12/10–6	1074	457	4267	0.01	0.04	13.4	21.0	330	373

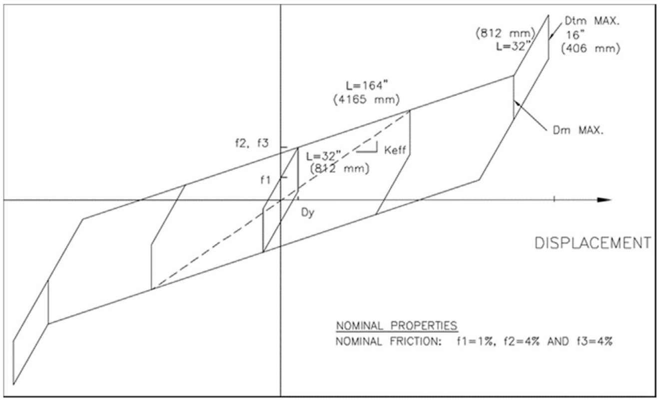


Fig. 29. Nominal Hysteresis Curve for FPT-8836/22-20/16-8 triple pendulum isolator (after EPS Inc.).

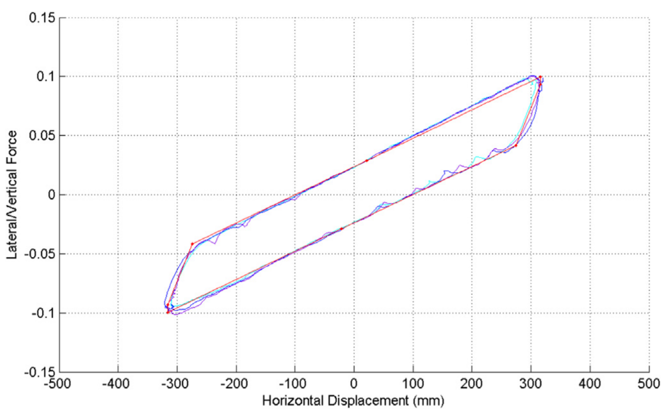


Fig. 30. Harmonic three-cycle tests on a FPT8836/22-20/16–8 prototype (Vertical Load varied between 21.4 − 19.6 MN, Maximum Lateral Load = +2.1 − 2.1 MN and Average maximum displacement = 312 mm).

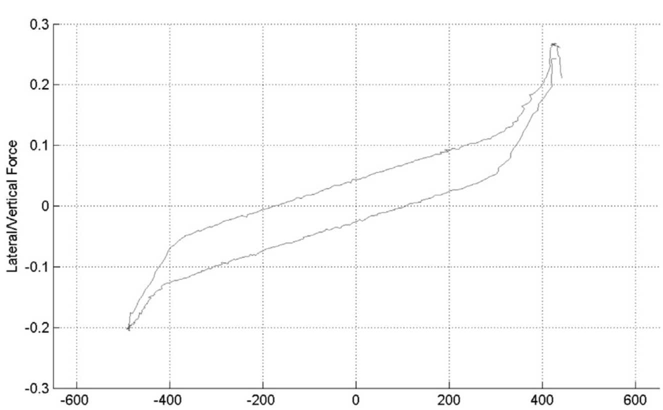


Fig. 31. Shear Load Reserve Capacity Test (Harmonic one-cycle tests on a FPT8836/22-20/16-8 prototype (Vertical Load varied between 10.0 and 11.6 MN, Maximum Lateral Load = +3.1 to −2.2 MN, Maximum displacement = +423 to −480 mm).

and shear load capacity. Fig. 31 (compare with Fig. 29) illustrates the one cycle force-displacement loop that verifies the shear load capacity. As it can be assessed the nominal design shear force capacity of 10% of the seismic weight shown in Fig. 30 can be as much as 20% as indicated in Fig. 31. Note that the ordinate of these plots indicate the ratio of the lateral load to the axial load.

The location of the isolator units in the isolation interface is illustrated in Fig. 32.

Table 15 indicates results obtained from the equivalent lateral force (ELF) analysis, based on the weighted average of upper and lower bound isolator properties. The weighing depends on the number of isolators used from each type. The Lower Bound analysis controlled the isolator displacements while the Upper Bound analysis controlled the interstory drifts, frame demands, and floor accelerations. Average over the set of ground motions of the nonlinear response history analysis (NLRHA) results are summarized in Table 16. In these tables K_{eff} and ξ_{eff} denote the effective values of lateral stiffness and equivalent damping ratıo. W denotes the seismic weight of the superstructure.

As per ASCE 7–10 [3] code, the minimum base shear under DBE is $0.9 \times 0.07\,W = 0.063\,W$, where W is the seismic weight and the minimum displacement demand under MCE is $0.8 \times 34.03 = 27.22$ cm. Thus the ELF results control the minima for the forces. Based on these results and considering ASCE 7–10 [3] code minima; a base shear of 0.063 W has been assumed for DBE and a displacement of 29.47 cm has been considered for MCE. Table 17 summarizes the values used for design.

An important demand parameter for the evaluation of likely performance of structural elements and other deformation-sensitive components is peak inter-story drift ratio. Figs. 33, 34 illustrate respectively the height-wise profile of story displacement and story drift ratios at DBE level ground motion, based on the average results of f horizontal ground motions applied in two orthogonal directions. Similarly Figs. 35, 36 respectively illustrate the height-wise profile of story displacement and inter-story drift ratios at DBE level ground motion, based on application of orthogonal horizontal ground motions. As it can be assessed the inter-story drift ratios are limited by almost 0.05 and always under 0.01 as expected per the performance objectives.

Another important aspect of seismic performance assessment is potential damage to acceleration-sensitive non-structural systems. Seismic isolation is often used to reduce floor accelerations in hospitals for the protection of sensitive medical equipment, as well as ceilings, parapets, and unanchored or lightly anchored architectural systems. On the other hand, a relatively tall and flexible structure on top of the isolation system may also lead to higher floor accelerations. In evaluating damage to nonstructural components, not only the peak floor acceleration at each floor, but also the peak floor spectral acceleration over a frequency range consistent with expected natural frequencies of nonstructural components, needs to be considered.

Fig. 37 displays the floor spectra in X direction at 10th floor associated with seven sets of scaled DBE level ground motion. Average floor spectra is indicated with black color. The peak horizontal floor accelerations vary almost linearly starting from 0.1 g at the ground floor and increasing to 0.2 g at the 10th floor. Floor response computations were carried out under DBE level ground motion and using nominal properties of the isolators.

As of May 2017, the Adana Integrated Health Campus has started to function. Fig. 38 shows the parking floor with isolators installed at the top of the columns. Fig. 39 illustrates an aerial view of the hospital complex.

7. Conclusions and suggestions

Seismic isolation technology is being implemented in Turkey at an accelerated rate, especially after the 1999 Kocaeli earthquake. The enforcement of Ministry of Health for the use of seismic isolation for hospitals in medium-to-high seismic hazard regions is an important and rational decision.

Considering the amount of investment in health sector and high level of seismicity in Turkey, seismic isolation appears to be the only solution to achieve the performance objective of being “operational / functional” for the medical services as well as the investment. In this connection, the seismic safety level of both the structure and non-

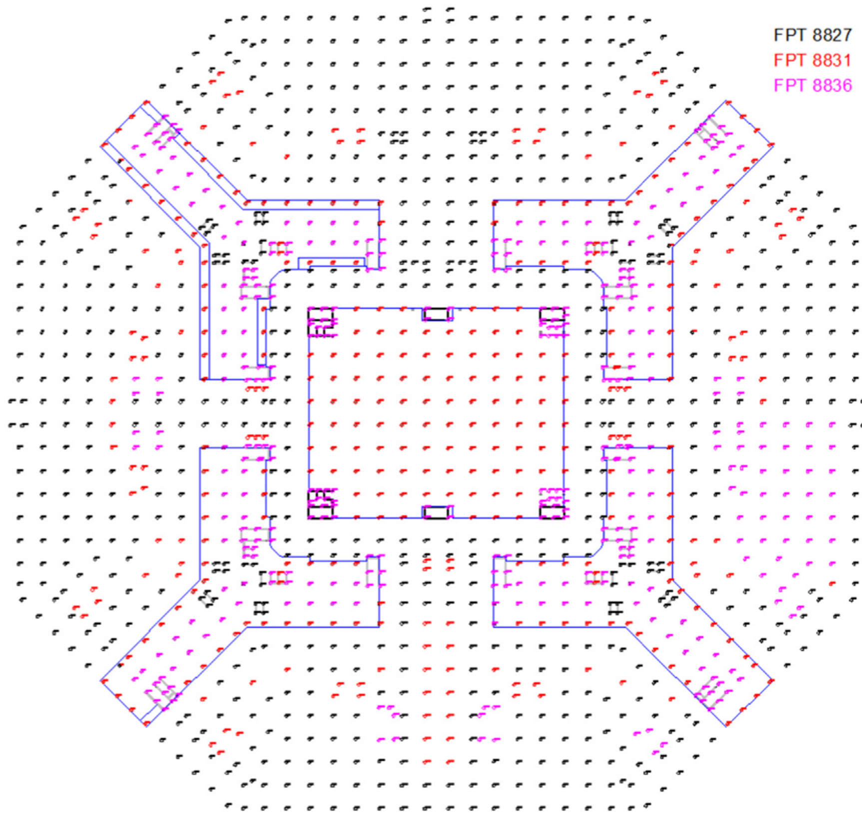


Fig. 32. Layout of the isolator units in the isolation interface level.

Table 15
ELF analysis results.

Parameter	DBE	MCE
K_{eff} (kN/m)	5968.49	2430.98
ξ_{eff}	0.44	0.16
T_{eff} (s)	2.27	3.56
Shear (% of W)	0.07	0.11
Displacement (cm)	8.43	34.03

Table 16
NLRHA Results.

Parameter	DBE	MCE
Shear (% of W)	0.06	0.09
Displacement (cm)	7.65	29.47

structural content of the existing hospitals should also be upgraded so that they could serve after a big earthquake.

Even though the design and construction of structures with seismic isolation involve quite complicated procedures that require advanced engineering input, it has been effectively implemented in important world-class projects in Turkey. The new official code prepared for the seismic isolation design for buildings will certainly regulate as well as encourage the new applications.

As is the case with new engineering applications, it is inevitable that some problems occur during the design and construction process. These problems can be mainly divided into two groups as “technical” and “logistics”.

7.1. Technical problems

The technical problems can be listed as follows:

i. Number of experienced engineers for the analysis and design of the seismically isolated structures is quite limited in Turkey.
ii. Similarly, number of architects who has experience in providing solutions in seismically isolated buildings is also quite limited in Turkey.
iii. The official national code is only very recently available in Turkey to regulate the analysis and design of seismically isolated structures.
iv. In the application process, the number of contractors who have the experience of constructing seismically isolated structures are quite limited in Turkey

Table 17
Selected design values.

Parameter	ELF	NLRHA	CRITERIA	SELECTED VALUE
Shear under DBE (% of W)	0.07	0.06	Greater of 90% of ELF value or the NLRHA value	0.063
Displacement under MCE (cm)	34.03 cm	29.47 cm	Greater of 80% of ELF value of the NLRHA value	29.47 cm

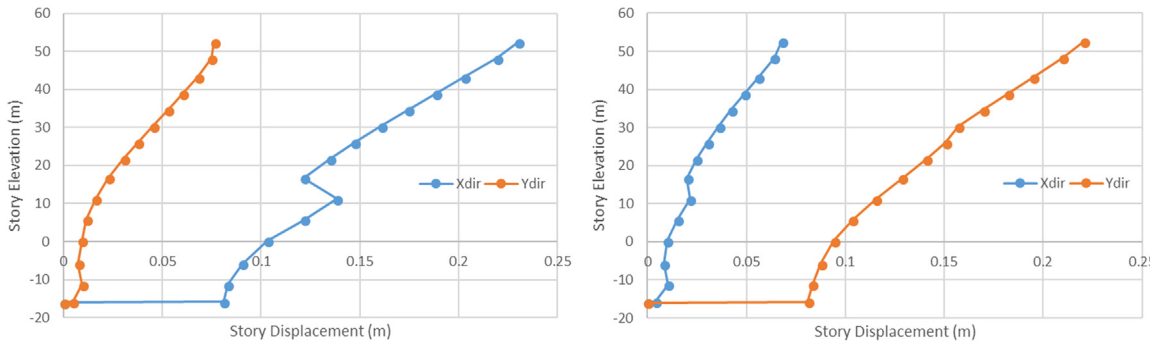


Fig. 33. DBE level story displacements (average of 7 NLRHA results).

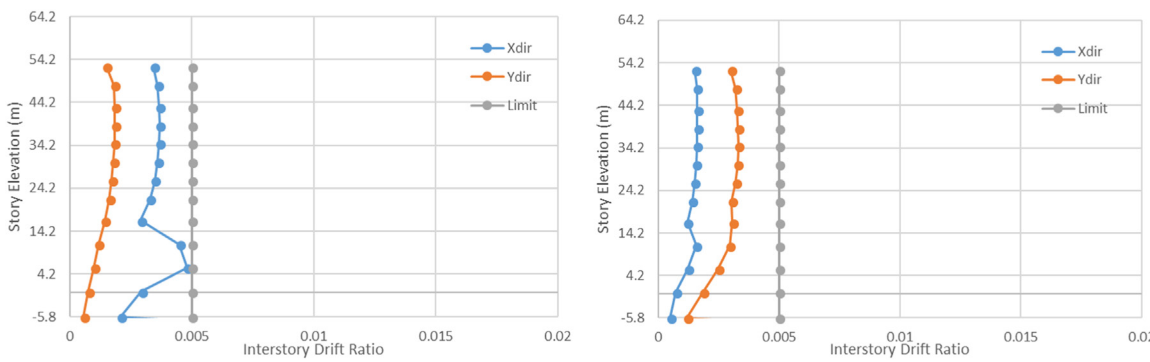


Fig. 34. DBE level inter-story drift ratios above the isolation level (average of 7 NLRHA results).

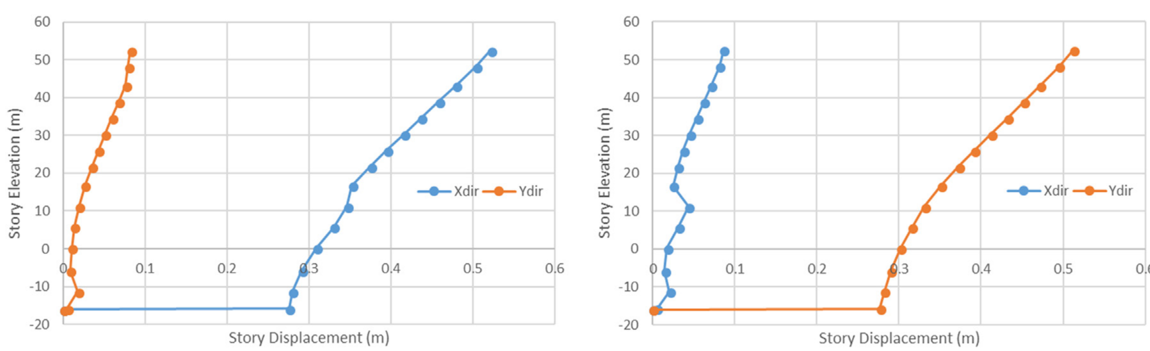


Fig. 35. MCE level story displacements (average of 7 NLRHA results).

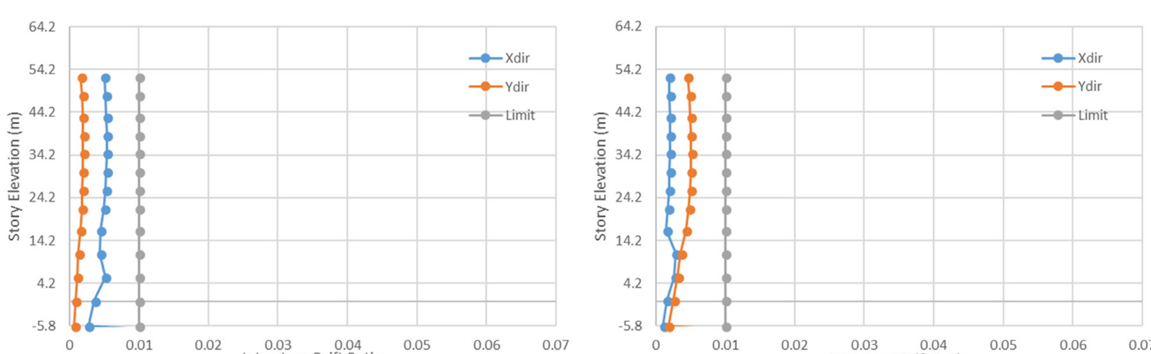


Fig. 36. MCE level inter-story drift ratios (average of 7 NLRHA results).

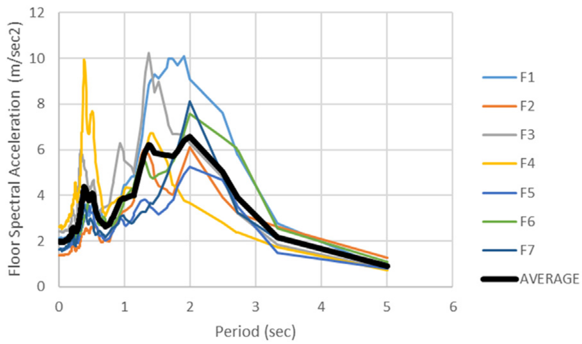


Fig. 37. X direction Floor spectra at 10th floor associated with seven sets of scaled DBE level ground motion (Black color indicates the average of these spectra).

7.2. Logistic problems

The logistic problems can be listed in the following:

i. The isolator production is emerging. However, currently, most of the isolator manufacturers are located outside Turkey.
ii. The testing facilities, with capability to provide real time testing to large scale isolators, are few in number and located outside the country. Testing process of the isolation units is a vital issue that requires the preparation of an official regulation concerning the selection and licensing of testing facilities.
iii. Lack of knowledge, in investors, contractors and engineers, about the selection, procurement, testing and installation stages of the seismic isolation process results in delays in construction schedule.

7.3. General aspects to be considered for effective design

In general, it should be kept in mind that the appropriate use of seismic isolation technology requires the adherence to the following points. Otherwise, the seismic safety of the isolated structure may possibly be lower than that of the conventional one.

i. A rational and functional architectural design, especially for hospitals.
ii. A reliable definition of the seismic input (Rational incorporation of near fault ground motion effects, long period and long duration ground motions characteristics in the design basis ground motion is of importance).
iii. A dependable, robust and resilient design using recent design codes, in conformity with the rational performance objectives and with appropriate peer review.
iv. A careful selection, design, manufacturing, testing, installation, protection and maintenance of the seismic isolation units,

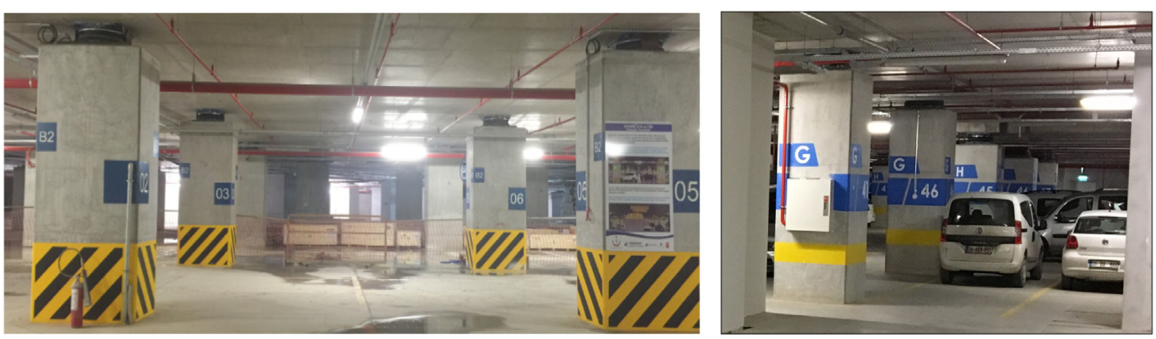


Fig. 38. Parking floor of the hospital complex with isolators installed at the top of the columns.

Fig. 39. An aerial view of the Adana Integrated Health Campus.

v. A good construction implementation, with quality assurance and control and with particular attention to be paid to seismic joints, and lifelines crossing the isolation interface.
vi. A minimum strong motion instrumentation installed in the base isolated structure to understand the earthquake response of seismic isolation systems,

Furthermore, the training of engineers for the proper and correct utilization of seismic isolation techniques, as well as licensing, needs to be considered for the healthy development of applications.

Finally, as required by the new code, the peer review process needs to be structured and become an integral part of the hazard assessment, design and implementation of the seismic isolation applications.

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References

[1] AASHTO. Guide specifications for seismic isolation design. 3rd edition Washington, DC: American Association of State Highway and Transportation Officials (AASHTO); 2010.
[2] ACI 209, Guide for modeling and calculating shrinkage and creep in hardened concrete. Reported by ACI Committee 209. Farmington Hills, MI, USA; 2008.
[3] ASCE 7-10. Minimum design loads for buildings and other structures, standard. Reston, Va: American Society of Civil Engineers; 2010.
[4] ACI 318-11. Building code requirements for structural concrete and commentary. ACI Committee 318. Farmington Hills, MI, USA; 2011.
[5] ASCE 41-13. Seismic evaluation and retrofit of existing buildings. Reston, Va: American Society of Civil Engineers; 2014.
[6] ASCE 7-16. Minimum design loads for buildings and other structures, standard. Reston, Va: American Society of Civil Engineers; 2016.
[7] Asher JW, Young RP, Ewing RD. Seismic isolation design of the Arrowhead Regional Medical Center. Tall Spec Build 2001;10(5):321–34. [December 2001].
[8] Bertero RD, Bertero VV. Performance-based seismic engineering: the need for a reliable conceptual comprehensive approach. Earthq Eng Struct Dyn 2002;31(3):627–52. [2002].
[9] Boroschek K, Retamales R. Base isolation in hospitals, WHO Collaborating Center Disaster Mitigation in Health, Seminar on Design of Health Facilities to Resist Natural Hazards; September 30 – October 1, 2002.
[10] Briseghella B. Lift-up and base isolation as a retrofit technique for R.C. existing building, In: Proceedings of the 15th WCEE. Lisbon, Portugal; 2012.
[11] BSL. Building Standard Law of Japan ⟨www.bcj.or.jp⟩; 2013.
[12] Buckle G. Passive control of structures for seismic loads. In: Proceedings of the 12th WCEE 2000. Wellington, NZ; 2000.
[13] Castellano MG, Cosenza E, Di Sarno L, Guerriero L, Lupoi G., Infanti S. Seismic isolation of hospitals: the examples of Udine, Lisbon and Naples Hospitals. In: Proceedings of the 10th world conference on seismic isolation, energy dissipation and active vibration control of structures. Istanbul, Turkey; 2007.
[14] Castellano MG. Italian experience in seismic retrofit of buildings through seismic isolation or energy dissipation. Proceedings, SMAR 2015; 2015.
[15] Clark PW, Higashino M Kelly JM. Performance of seismically isolated structures in the January 17, 1994 Northridge Earthquake. UC Berkeley, CA; 1995.
[16] Constantinou MC, Kalpakidis I., Filiatrault A, Ecker RA. LRFD-based analysis and design procedures for bridge bearings and seismic isolation, MCEER 11-0004; 2011.
[17] Demircioğlu Mine B, Şeşetyan ve K. İhtimal Hesapları Teorisine Bağlı Sismik Tehlike Hesaplarının Yapılması, Coğrafi Bilgi Sistemleri Ortamında Eşdeğer Spektral İvme Haritalarının Çıkartılması, Türkiye Sismik Tehlike Haritasının Güncellenmesi Projesi (UDAP-Ç-13-36), Boğaziçi Üniversitesi; 2015.
[18] DLH. Kıyı ve Liman Yapıları, Demiryolları ve Hava Meydanları Inşaatlarına Ilişkin Deprem Teknik Yönetmeliği (Seismic Design Code for Construction of Railways. Harbors and Airports), Ulaştırma Bakanlığı, Ankara; Ağustos 2007a.
[19] Di Sarno L, Cosenza E., Pecce MR. Application of base isolation to a large hospital in Naples, Italy. In: Proceedings of the 10th world conference on seismic isolation, energy dissipation and active vibration control of structures. Istanbul, Turkey; 2007.
[20] Di Sarno L, Yenidogan C, Erdik M. Field evidence and numerical investigation of the Mw7.1 October 23 van, Tabanli and the Mw5.7 November earthquakes of 2011. Bull Earthq Eng 2013;2013(11):313–46.
[21] EC-8. Eurocode 8, Design of structures for earthquake resistance Part 1: general rules, seismic actions and rules for buildings. EN 1998-1:2005. Comitee Europeen de Normalisation, Brussels; 2005.
[22] EN 15129. Antiseismic devices. brussels: comité Européen de normalisation; 2009.
[23] Erdik M., Tüzün C, Ülker Ö. Evaluation of seismic isolation applications of health care facilities in Turkey. In: Proceedings of 14WCSI, San Diego, California; 2015a.
[24] Erdik M. State of the art on application, R&D and design rules for seismic isolation and energy dissipation for buildings, bridges and viaducts, cultural heritage and chemical plants in Turkey. Invited Paper. In: Proceedings of the 15th WCSI and NZSEE 2017, Wellington, NZ; 2017.
[25] Erdik M, Demircioglu MB, Sesetyan K, Harmandar E. Characterization of long period strong ground motion. J Seismol Earthq Eng 2011.
[26] Erdik M. State of the art on application, R&D and design rules for seismic isolation and energy dissipation for buildings, bridges and viaducts, cultural heritage and chemical plants in Turkey. In: Proceedings 14th WCSI San Diego, CA; 2015.
[27] Erdik M, Tüzün C, Ülker Ö. Evaluation of seismic isolation applications of health care facilities in Turkey, In: Proceedings 14th WCSI San Diego, CA; 2015b.
[28] Erdik M, Demircioğlu MB, Şeşetyan K, Harmandar E. Earthquake hazard assessment for adana hospital. Boğaziçi Üniversity, Istanbul, Turkey; 2013.
[29] Erdik M, Tuzun C, Ulker O. Evaluation of seismic isolation applications of health care facilities in Turkey. In: Proceedings of the 14th world conference on seismic isolation, energy dissipation and active vibration control of structures. San Diego (CA), USA; 2015c.
[30] Erdik M. Report on 1999 Kocaeli and Düzce (Turkey) earthquakes, in Structural Control for Civil and Infrastructure Engineering; 2001. p. 149–86.
[31] Erdik M, Kamer Y, Demircioglu M, Sesetyan K. 23 October 2011 Van (Turkey) Earthquake. Nat Hazards (2012) 2012;64:651–65.
[32] DLH. Kıyı ve Liman Yapıları, Demiryolları ve Hava Meydanları Inşaatlarına Ilişkin Deprem Teknik Yönetmeliği (Seismic Design Code for Construction of Railways, Harbors and Airports). Ulaştırma Bakanlığı, Ankara, Ağustos 2007b.
[33] Fenz DM, Constantinou MC. Development, Implementation and Verification of Dynamic Analysis Models for Multi-Spherical Sliding Bearings. Buffalo, NY: Multidisciplinary Center for Earthquake Engineering Research; 2008.
[34] Guisasola A. How base isolation benefits the architectural design of hospital buildings, In: Proceedings of the 15th WCEE, Lisboa; 2012.
[35] Kubin D, Kubin A, Özmen OB, Sadan, Eroğlu E, Sucuoğlu H. Seismic retrofit of an existing multi-block hospital by seismic isolators. In: Proceedings of the 15th world conference on earthquake engineering, Lisboa, Portugal.; 2012.
[36] Martelli A, Clemente P, De Stefano A, Forni M, Salvatori A. Recent development and application of seismic isolation and energy dissipation and conditions for their correct use. Perspectives on European Earthquake Engineering and Seismology. 1. A.Ansal, Springer; 2014.
[37] Masuzawa Y, Hisada Y. Current state of retrofitting buildings by seismic isolation in Japan. In: Proceedings of the 15th world conference on earthquake engineering; 2012.
[38] Masuzawa Y, Hisada Y. Seismic isolation retrofit of a medical complex by integrating two large-scale buildings, In: Proceedings of the 14th WCEE. Beijing, China; October 12–17 2008.
[39] McWitty W, Constantionou MC. Property modification factors for seismic isolators: design guidance for buildings, are considered. MCEER 15-0005; 2015.
[40] Nagarajaiah S, Xiaohong S. Response of base-isolated USC hospital building in Northridge earthquake. J Struct Eng 2000;126:1177.
[41] NTC-08. Norme tecniche per le costruzioni- DM 14-01-2008, Italy; 2008.
[42] Otani Shunsuke. Earthquake resistant design of reinforced concrete buildings: past and future. J. Adv. Concr. Technol. 2004;2(1). Japan Concrete Society.
[43] Otani S, Kani N. Japanese state of practice in design of seismically isolated buildings. In: Proceedings of the 4th US- Japan workshop on performance-based earthquake engineering methodology for reinforced concrete building structures. Toba, Japan; 2002.
[44] Priestley MJN, Kowalsky MJ. Direct displacement-based seismic design of concrete buildings. Bull NZ Natl Soc Earthq Eng 2000;33(4):421–44.
[45] Priestley MJN, Calvi GM, Kowalsky MJ. Displacement-based seismic design of structures. IUSS Press, Pavia, Italy; 2007. p. 270.
[46] Shibata A, Sozen M. Substitute structure method for seismic design in reinforced concrete. J Struct Div – ASCE 1976;102(12):3548–66. [1976].
[47] TASI. Draft earthquake resistant design code for seismic isolation (Submitted to the Ministry of Public Works). Turkish Association for Seismic Isolation; 2008.
[48] TSDC. Specification for buildings to be built in seismic zones. Ministry of Public Works and Settlement Government of Republic of Turkey. Ankara, Turkey; 2007.
[49] Yashinsky M. Earthquake damage to structures Ch.18. In: Chen W-F, Lui EM, editors. Handbook of structural engineering. CRC Press; 2005.
[50] Yenidogan C, Erdik M. A comparative evaluation of design provisions for seismically isolated buildings. Soil Dyn Earthq Eng 2016;90:265–86.
[51] TERDC. 2018 Revision of the Turkish seismic design code. The Disaster and Emergency Management Presidency of Turkey. Ankara; 2018.
[52] Thompson A, Mackenzie C. Aspects of design of the base isolated Christchurch Hospital acute services building. In: Proceedings of structural engineering society of New Zealand conference, 2012. Wellington, NZ; 2017.