ДГКС ASES | E-25
Kongres 2026
Congress 2026

doi: 10.67563/ASESCONF2026021S

Jovan Starčević[1], Marko Marinković[2]

SEISMIC ANALYSIS OF A DAMAGED RC TUNNEL FORM BUILDING IN TÜRKIYE

Summary: This paper analyses the seismic behaviour of a RC building with a tunnel structural system under seismic loading. This system is increasingly used in modern construction due to spatial rigidity and rapid construction; however, strong earthquakes, such as the one in Türkiye in 2023, have shown that even such buildings can sustain significant damage. An existing building designed according to the Turkish Earthquake Code TEC2007 was studied, with results compared to the newer TBSC2018 regulation and a response spectrum from a real earthquake record. Different structural models were considered depending on code and load combinations, including variations of the behaviour factor in TEC2007 and wall thickness. Analyses were performed using SAP2000. Particular attention was given to reinforcement of walls and coupling beams and to construction quality. Base isolation with lead-rubber bearings was also considered as a possible improvement.

Keywords: earthquake, RC tunnel form building, multimodal spectral analysis, coupling beams, base isolation

ANALIZA SEIZMIČKOG PONAŠANJA OŠTEĆENE ARMIRANOBETONSKE ZGRADE TUNELSKOG SISTEMA U TURSKOJ

Rezime: Rad analizira ponašanje armiranobetonske zgrade tunelskog sistema pri dejstvu seizmičkih opterećenja. Ovaj konstruktivni sistem sve je zastupljeniji u savremenoj gradnji zbog prostorne krutosti i brzine izvođenja, ali su snažni zemljotresi, poput onog u Turskoj 2023. godine, pokazali da i ovakvi objekti mogu pretrpeti značajna oštećenja. Analizirana je postojeća zgrada projektovana prema turskom propisu TEC2007, uz poređenje rezultata sa novijim propisom TBSC2018 i spektrom dobijenim iz realnog zapisa zemljotresa. Razmatrani su različiti modeli konstrukcije u zavisnosti od propisa i kombinacija opterećenja, sa promenom faktora ponašanja u modelu po TEC2007 i debljine zidova. Proračuni su sprovedeni u programu SAP2000. Posebna pažnja posvećena je armiranju zidova i veznih greda, kao i uticaju kvaliteta izvođenja. Kao mogućnost poboljšanja razmatrana je primena bazne izolacije sa elastomernim ležištima sa olovnim jezgrom.

Ključne reči: zemljotres, armiranobetonski tunelski sistem konstrukcije, multimodalna spektralna analiza, vezne grede, bazna izolacija

[1] STRABAG doo, jovanstarcevic99@gmail.com
[2] Faculty of Civil Engineering, University of Belgrade, Serbia, mmarinkovic@grf.bg.ac.rs

1. INTRODUCTION

Türkiye is located on the Anatolian tectonic plate, which is bounded by the Eurasian, African, and Arabian plates. The interaction of these plates generates significant seismic activity, primarily along the North Anatolian Fault (NAF) and the East Anatolian Fault (EAF), making Türkiye one of the most seismically active regions worldwide.[1]

Throughout recent decades, destructive earthquakes, including the 1999 Izmit, 2011 Van and 2020 Izmir earthquakes, have demonstrated the importance of appropriate seismic design, construction quality, and reinforcement detailing. The most severe recent event occurred on 6 February 2023, when two major earthquakes with magnitudes Mw 7.8 and Mw 7.5 affected southern Türkiye and northwestern Syria along the East Anatolian Fault. The recorded ground motions exceeded design expectations in several locations, with peak ground acceleration values approaching 1.3g, resulting in extensive structural damage and more than 55,000 fatalities.

The observed failures were associated with both extreme seismic demand and structural deficiencies, including inadequate confinement reinforcement, insufficient lap splice lengths, poor reinforcement continuity, irregular configurations, and construction-related issues. These deficiencies significantly reduced the ductility and seismic performance of reinforced concrete buildings. The building investigated in this study suffered damage during the 2023 earthquake sequence and is analysed as a case study to identify the main factors influencing its observed structural response.

Tunnel form construction is widely used in seismic-prone regions, including Türkiye, due to its rapid construction process and favourable stiffness and strength characteristics. The system consists of monolithically cast reinforced concrete walls and slabs, where the lateral resistance is primarily provided by closely spaced shear walls. Despite its advantages, inadequate detailing and construction practices may significantly affect its seismic performance. Given these characteristics and its widespread application in Türkiye, the tunnel formwork system represents a highly relevant structural typology for seismic assessment studies.

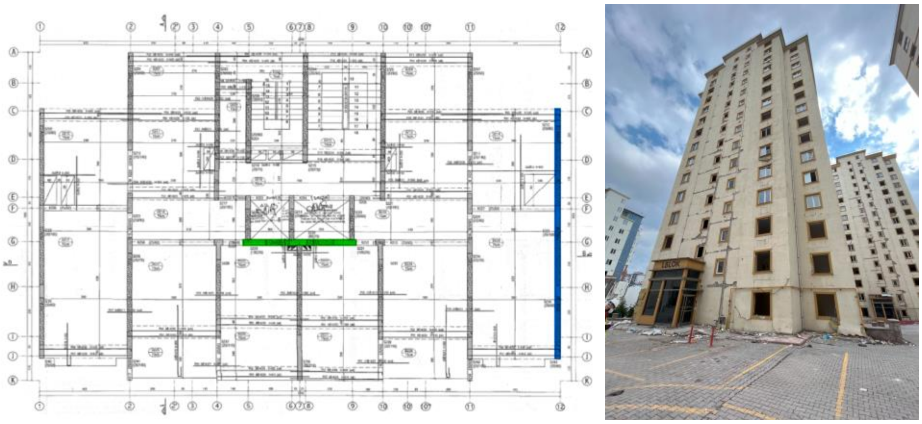
Accordingly, this paper focuses on a reinforced concrete building constructed using the tunnel form system that experienced significant damage during the 2023 Kahramanmaraş earthquake sequence. The study aims to investigate the observed damage mechanisms, relate them to the seismic demand and structural characteristics of the system, and evaluate the seismic performance of the building within the context of contemporary seismic design principles.

2. DESCRIPTION OF THE CASE STUDY BUILDING

The structure investigated in this paper is a tunnel form residential building located in Dulkadiroğlu Aslanbey, Türkiye. The study examines the seismic performance of the building, identifies the main structural deficiencies observed after the 2023 earthquake, and discusses potential measures to improve the seismic performance of similar tunnel form buildings.

The building has a predominantly regular and symmetric plan, except for the elevator core and stairwell. It consists of a basement, ground floor, 14 typical storeys, and a roof level (BS+G+14+R), with a total height of approximately 47 m and plan dimensions of 23.5 × 15.5 m. The building is supported by a reinforced concrete foundation grillage with tie beams, providing load transfer from the wall system to the supporting soil. The lateral load-resisting system comprises reinforced concrete shear walls connected by coupling beams. Walls are 20 cm thick in the Y-direction and 25 cm thick at the elevator core in the X-direction, while the floor slabs are 15 cm thick reinforced concrete elements. Concrete class C30/37 and reinforcement steel grade S420 were adopted throughout the structure. Reinforcement detailing follows typical tunnel form construction practice, with mesh reinforcement distributed along the height of the structure and additional reinforcement provided around openings.

Structural information and damage documentation were obtained through post-earthquake field investigations. The observed damage included concrete spalling, reinforcement buckling, diagonal cracking in coupling beams, and damage concentrated in the boundary regions of shear walls. These failures are attributed to inadequate reinforcement detailing, including insufficient concrete cover, inadequate lap splice lengths, limited reinforcement continuity, and insufficient boundary confinement, which reduced the ductility and seismic performance of the structure. The following figures show the collected documentation and photographs of the investigated building.



a), b)

c), d), e)

Figure 1: Case study building: a) typical floor plan; b) exterior view; c, d, e) representative earthquake induced damage [3]

3. NUMERICAL MODELS

3.1. STRUCTURAL MODELING

The case study building was modelled in SAP2000. Reinforced concrete shear walls, floor slabs, and coupling beams were discretised using shell finite elements. Material properties were defined according to concrete class C30/37, while a linear-elastic constitutive model was adopted for the purpose of modal and response spectrum analyses.

The structure was assumed to have a fixed-base condition. Floor diaphragms were considered rigid in their in-plane behaviour, ensuring an adequate transfer of lateral seismic forces to the vertical load-resisting elements. Wall openings were not explicitly modelled, given their limited influence on the global dynamic response of the structure.

The lateral load-resisting system is primarily composed of reinforced concrete shear walls interconnected by coupling beams, forming a box-type structural behaviour typical of tunnel formwork systems. The three-dimensional finite element model used for the analysis is presented in Figure 2.

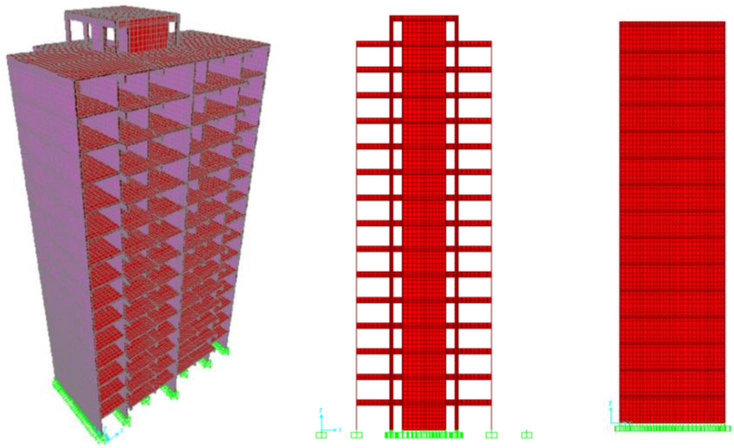


Figure 2: 3D software model of Tunnel form building in SAP

3.2. SEISMIC LOAD COMBINATIONS – TURKISH CODES

For the purposes of structural analysis and seismic response evaluation, Turkish seismic design codes were considered. Seismic load combinations defined in the Turkish Earthquake Code 2007 (TEC2007) and the Turkish Building Seismic Code 2018 (TBSC2018) were adopted. The building was designed in 2017 according to TEC2007, where reinforced concrete wall systems are classified as normal ductility system with R=4. An additional analysis with R=6 was performed for comparison with other seismic code models.

According to TEC2007, the seismic load combination is 1.0G + 0.3Q+ E, while TBSC2018 define three combinations: 1.0G + 0.3Q + E, 0.9G + E and 1.0G + 1.0Q + E, where G represents permanent loads, Q variable (live) loads, and E the seismic action. [4,5]

Seismic action is considered in two orthogonal directions (Ex and Ey). Directional combinations follow the 100%–30% rule, leading to ±Ex ± 0.3Ey for the X-direction and ±0.3Ex ± Ey for the Y-direction, with both positive and negative signs considered, resulting in eight load cases. Response spectrum analysis is performed using code-based spectra. In addition, a spectrum derived from recorded ground motion (station TK4621) is also considered, using the same combination rules as TBSC2018. Envelope functions are applied to obtain governing response values. A gravity load combination of 1.35G + 1.0Q is also included. The mass source definition includes permanent and variable loads, consistent with the dynamic analysis requirements.

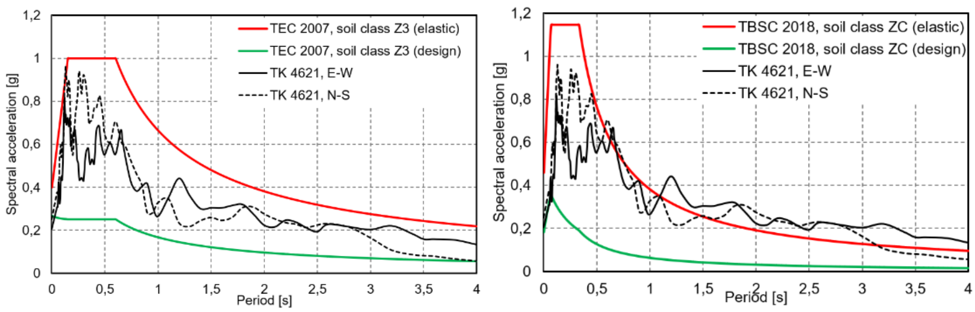


Figure 3: a) Horizontal elastic response spectrum; b) Horizontal design response spectrum

4. NUMERICAL ANALYSIS

4.1. MODAL ANALYSIS RESULTS

Modal analysis was performed considering the first 25 vibration modes, ensuring that the cumulative effective modal mass exceeds 90% of the total structural mass, in accordance with Eurocode 8 requirements. The seismic mass was defined considering the permanent loads and the appropriate contribution of variable loads according to seismic design provisions. The first mode is dominated by global translation in the X-direction, while the second mode corresponds to translation in the Y-direction. The third mode exhibits a torsional response, indicating the coupled translational-torsional behaviour of the shear wall system. The first three mode shapes are presented in Figure 3.

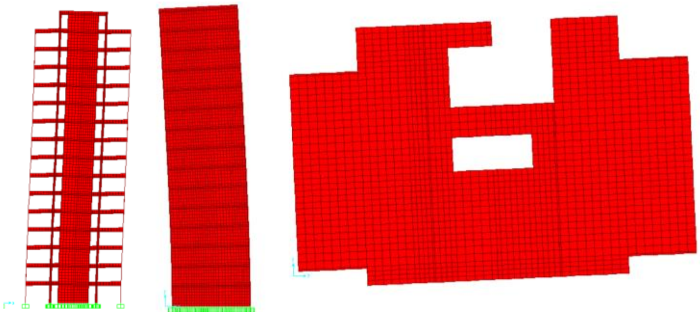


Figure 4: First Three Modes Shapes

4.2. MULTIMODAL SPECTRAL ANALYSIS

For the response spectrum analysis, the structure was subjected to its self-weight, an additional permanent load of 2.5 kN/m², and a live load of 3.0 kN/m², applied as uniformly distributed loads on all floor slabs. Soil type Z3 and an importance factor of 1.0 were adopted. According to the Turkish seismic codes, the behaviour factor is equal to 4 in TEC2007 and 6 in TBSC2018 for wall-dominated structural systems. To enable a consistent comparison between the considered models, all analyses were performed using a behaviour factor of 6, while an additional model based on TEC2007 with a behaviour factor of 4 was analysed separately.

Response spectrum analyses were performed using the spectra prescribed by TEC2007 and TBSC2018, as well as a spectrum derived from the recorded accelerogram of the 2023 earthquake. Elastic response spectra were used to evaluate interstory drifts, whereas design spectra were adopted for structural design. A damping ratio of 5% was assumed, and load combinations were defined in accordance with the Turkish seismic code. The resulting internal forces and stress distributions obtained from SAP2000 were used for the subsequent evaluation of the structural response.

The comparison of the obtained results indicates that the highest internal forces in the critical shear walls are obtained using the response spectrum defined by TEC2007. In contrast, the results obtained using TBSC2018 and the response spectrum derived from the recorded 2023 earthquake ground motion are generally comparable, with only minor differences observed.

The governing results from all considered seismic load combinations were used for the subsequent reinforcement design of the critical shear walls.

4.3. DESIGN OF CRITICAL SHEAR WALLS ACCORDING TO EUROCODE 8

Based on the analysis of internal forces, the critical shear walls in the X and Y directions were selected for design according to Eurocode 8 and Eurocode 2. The wall located along Grid G was identified as the governing wall in the X-direction, while the façade wall along Grid 12 exhibited the highest seismic demand in the Y-direction for all analysed models, as shown in Figure 1(a).

The selected walls were designed for combined axial load, bending moment, and shear. Flexural design was performed using interaction diagrams in accordance with Eurocode 2, while the shear design followed the provisions of Eurocode 8. [6,7]

4.3.1. Comparison of designed and existing reinforcement

According to the original design documentation, the shear walls were reinforced using welded wire meshes with additional reinforcement in the boundary regions. The results indicate that the reinforcement detailing was inadequate, contributing to the observed structural damage. The comparison with the calculated requirements shows that the reinforcement specified in the original design is generally lower, particularly in the boundary confinement regions of critical walls. In addition, insufficient lap splice lengths and discontinuity of vertical reinforcement between storeys were identified as significant deficiencies, reducing the expected ductility of the system and being consistent with the damage patterns observed after the earthquake. The analytical results also indicate significantly higher requirements for boundary confinement zones compared to those provided in the original design. The analytically obtained reinforcement was designed according to Eurocode 8 provisions for seismic walls. Optimization of shear reinforcement (stirrups and U-bars) was performed along the height of the structure, considering the variation of shear demand between storeys, which generally reduced the total reinforcement demand compared to the original design. In contrast, the original design employed horizontal reinforcement without optimization along the building height. Vertical reinforcement in the analytical model was designed as continuous bars extending over two storeys with adequate anchorage lengths. Overall, the differences between the calculated and original design reinforcement, particularly in boundary confinement, lap splice detailing, and reinforcement continuity, influenced the observed damage. Table 1 which presents the calculated reinforcement (both horizontal and vertical) indicates that the actual reinforcement is generally lower than the reinforcement required by the TEC2007model with R=4, which corresponds to the code applicable at the time of original design. In contrast, the reinforcement demands obtained from TEC2007 with R=6, TBSC2018 and the recorded earthquake spectrum are relatively similar.

Table 1: Story-wise reinforcement – X and Y direction wall

Floor	Total reinforcement per floor- wall in X directions - axis G[kg]				
	Actual	2007(R=4)	2007(R=6)	2018(G+Q+E)	TK4621(G+Q+E)
GF	1511,3	1678,1	1064,3	817	817
1st	1259,4	1434,1	795,6	508,1	600,9
2nd	1269,6	1087,3	795,6	508,1	600,9
3rd	1171,2	1087,3	620,4	508,1	508,1
4th	1153,2	1087,3	620,4	418,2	508,1
5th	1101,6	808,5	620,4	418,2	508,1
6th	1055,0	808,5	526,0	372	508,1
7th	991,4	751,3	526,0	372	418,2
8th	991,4	629,8	463,0	372	418,2
9th	991,4	548,8	463,0	372	372
10th	991,4	548,8	463,0	314,3	372
11th	991,4	463,0	388,7	314,3	314,3
12th	991,4	463,0	329,6	279,7	279,7
13th	991,4	329,6	270,5	256,6	256,6
14th	991,4	270,5	270,5	256,6	256,6
RF	991,4	246,8	246,8	233,5	233,5

Floor	Total reinforcemenet per floor- wall in Y directions - axis 12[kg]				
	Actual	2007(R=4)	2007(R=6)	2018(G+Q+E)	TK4621(G+Q+E)
GF	1024,0	1387,4	1006,5	908,6	908,6
1st	777,7	1387,4	698,8	566,3	505,7
2nd	752,6	959,4	565,5	505,7	505,7
3rd	804,0	959,4	565,5	505,7	505,7
4th	740,9	719,8	503,8	505,7	505,7
5th	740,9	719,8	503,8	505,7	505,7
6th	763,6	719,8	503,8	505,7	505,7
7th	746,0	670,4	503,8	505,7	505,7
8th	746,0	608,6	503,8	505,7	505,7
9th	746,0	608,6	503,8	505,7	505,7
10th	746,0	503,8	503,8	505,7	505,7
11th	746,0	503,8	503,8	505,7	505,7
12th	746,0	503,8	503,8	505,7	505,7
13th	746,0	503,8	503,8	505,7	505,7
14th	746,0	503,8	503,8	505,7	505,7

4.3.2. Effect of increased wall thickness on structural response

To better understand the structural behaviour, additional models with increased wall thickness (30 cm in the X-direction and 25 cm in the Y-direction) were developed for all load combinations and seismic codes. These models were introduced to investigate the influence of increased stiffness on the global structural response and the potential reduction in required reinforcement in critical regions. The analysis was performed at the ground floor level, where the most pronounced effects of stiffness modification were observed. Increasing the wall thickness reduced the natural vibration periods by approximately 5–10%, indicating higher global stiffness. Although the thicker walls attracted higher internal forces, the original models developed only 80–95% of these forces. At the same time, the boundary confinement zones were reduced by up to 20%, resulting in a lower reinforcement demand in the critical wall regions.

4.4. COUPLING BEAMS

4.4.1. General aspects

Coupling beams are structural elements that connect adjacent shear wall segments interrupted by openings for doors, windows, or vertical circulation. They ensure continuity of the wall system and significantly influence the global seismic response of coupled shear walls.

In seismic design, coupling beams contribute to stiffness, strength, and energy dissipation. Due to high shear demands and cyclic loading, they are often critical elements in the inelastic response of the structure. Their behaviour is characterized by moment reversals and significant shear demand, especially under higher mode effects.

Depending on design approach, coupling beams may be detailed with conventional longitudinal reinforcement or with diagonal reinforcement. Diagonal detailing is generally adopted when shear demands are high, as it provides improved ductility and energy dissipation capacity.

4.4.2. Design and comparison of governing coupling beams

The seismic demand was obtained from the Turkish seismic codes (TEC2007 and TBSC2018), while the reinforcement design of both shear walls and coupling beams was performed according to Eurocode 8 provisions. Envelope results from all considered seismic load combinations were used for design.

According to Eurocode 8, conventional detailing may be applied if at least one of the following conditions is satisfied: [7,8]

a) $V_{Ed} \leq f_{ctd} \cdot b_w \cdot h$ (1)

b) $\frac{l}{h} \geq 3$ (2)

Where is:

f_{ctd} – the design tensile strength of concrete for steel grade S420 is 1.61 MPa
h – the height of the beam
l – the length of the beam

Otherwise, diagonal reinforcement is required. In this paper, coupling beams are located along axis G in the X-direction and along axes 2 and 11 in the Y-direction. The X -direction beam is designed with diagonal reinforcement due to highest shear demands, while the Y-direction beam is detailed as conventional beams. The reinforcement design results obtained for the governing seismic load combinations of TEC2007, TBSC2018, and the recorded ground motion indicate a significantly higher reinforcement demand compared to the original design. In the available documentation, coupling beams were detailed using mesh reinforcement similar to shear walls, with only minor local strengthening around openings.

Although a direct quantitative comparison of reinforcement was not possible due to limited as-built data, the observed damage pattern indicates inadequate shear capacity and insufficient detailing in the existing beams. Beams designed according to Eurocode 8 show cases where shear demand exceeds the limits for conventional detailing, confirming that diagonal reinforcement would be required in several elements.

Diagonal reinforcement, while highly efficient in terms of ductility and energy dissipation, is still rarely adopted in practice in many regions, including Serbia, despite being incorporated in several international seismic design proviosions.

The observed diagonal cracking pattern is characteristic of shear failure under seismic loading, confirming insufficient reinforcement capacity and inadequate detailing in coupling beams.

5. APPLICATION OF BASE ISOLATION SYSTEM

Base isolation is one of the most effective modern seismic protection techniques, aiming to reduce the transfer of seismic energy from the ground to the superstructure. In buildings with basement levels, the isolation plane is located at the top of the basement, i.e. between the substructure and the superstructure. At this level, elastomeric bearings are installed, allowing controlled horizontal displacements while maintaining sufficient vertical load-bearing capacity. By decoupling the superstructure from the ground motion, the fundamental period of the structure is increased, resulting in a reduction of seismic forces and damage demands on structural elements. Several isolation systems can be applied, including elastomeric bearings, sliding bearings, and hybrid systems. In this study, lead rubber bearings (LRBs) were adopted due to their favourable balance between stiffness, energy dissipation, and self-centring capability. The analysed isolation system consists of MLRB-type bearings from the MAURER catalogue. The design of the bearings was based on the total structural mass, vertical load distribution, and the adopted number of bearings, ensuring adequate support of the primary shear walls. Since the TEC2007 model produced the highest vertical demands, it governed the bearing selection. Based on a total structural mass of 4250 t and 39 bearings, the average vertical reaction per bearing was calculated as 2135 kN. Additional criteria included a target displacement of 0.217 m and an effective horizontal stiffness of 1088 kN/m. According to these requirements, the MLRB-S-03-300 bearing (soft type) was selected, providing an optimal balance between seismic force reduction and displacement control.

The isolation devices were positioned at the interface between the basement and superstructure, beneath the primary shear walls, ensuring adequate transfer of vertical loads. Since the foundation system was not explicitly modelled, the isolation system was represented in SAP2000 using nonlinear spring (link) elements at the ground floor level. Horizontal stiffness values were assigned to degrees of freedom u1 and u2, while vertical stiffness was assigned to u3 according to the properties of the selected bearing and design calculations.

The application of base isolation in tunnel form buildings requires additional consideration due to the presence of continuous vertical reinforcement within the shear walls. Unlike conventional frame structures, where isolation devices can be more easily introduced between structural levels, tunnel form systems require modification of the wall force-transfer mechanism and appropriate detailing of transition and anchorage regions above the isolation interface. These details are necessary to ensure reliable transfer of vertical and horizontal forces while maintaining the required reinforcement continuity. Although this increases construction complexity and requires careful planning of the isolation level, base isolation remains a practical strengthening solution for stiff wall-dominated systems, as it reduces seismic demands and enables the superstructure to remain primarily within the elastic range during strong earthquakes.

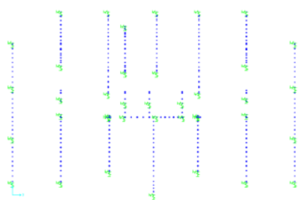


Figure 5: Location of base isolation bearings in the SAP2000 model

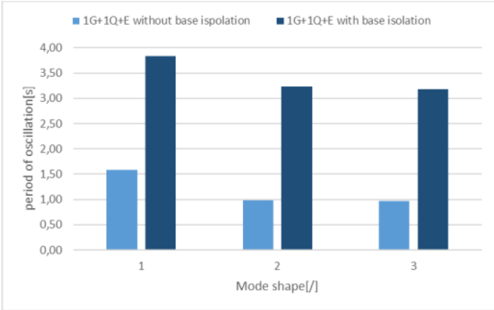


Figure 6: Comparison of the first three mode shapes for models with the load combination 1.0G+1.0Q+E, with and without base isolation

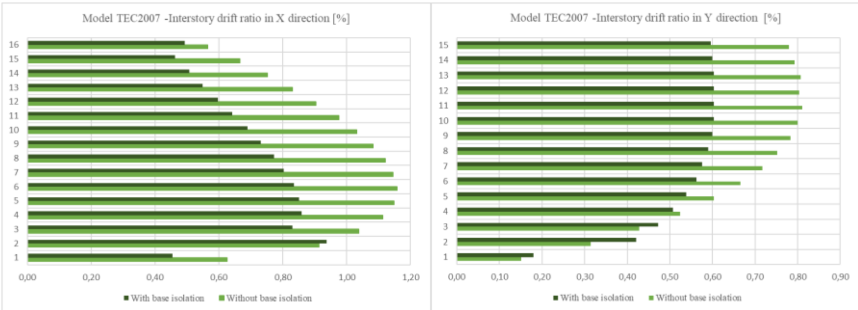


Figure 7: Comparison of interstory drift for wall in X and Y directions for model TEC2007 without and with base isolation

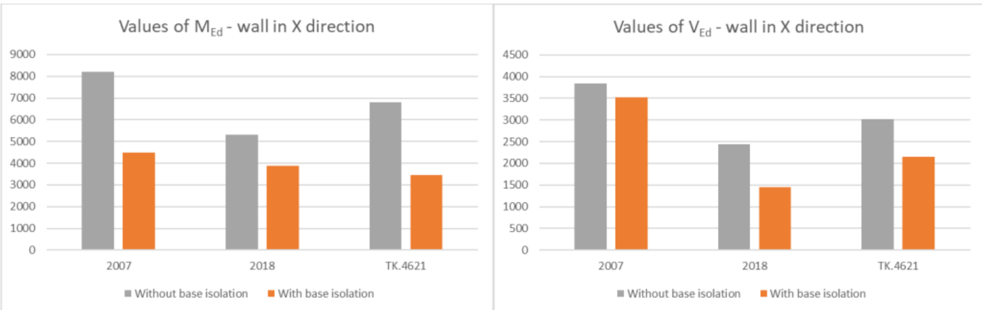


Figure 8: Comparison of M_{Ed} and V_{Ed} for the wall in the X direction without and with base isolation

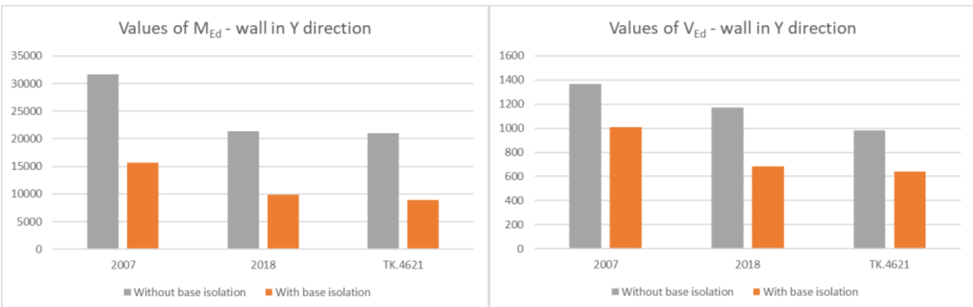


Figure 9: Comparison of M_{Ed} and V_{Ed} for the wall in the Y direction without and with base isolation

6. CONCLUSION

Tunnel-form reinforced concrete buildings represent a widely used structural system in seismic regions due to their high in-plane stiffness, regularity, and construction efficiency. However, recent seismic events, including the 2023 Türkiye earthquake, have demonstrated that even such systems may experience severe damage when design assumptions and construction quality are not adequately satisfied. The investigated building from this study represents one such case.

The structure was originally designed according to the 2007 Turkish seismic code and was additionally assessed using the 2018 Turkish Earthquake Code and the recorded ground motion from the 2023 event. Comparative analyses indicated that the original design assumptions, particularly regarding reinforcement detailing and ductility level, led to a generally lower required reinforcement ratio compared to the as-built condition. Significant discrepancies were observed in boundary zones and confinement regions, where insufficient detailing, inadequate anchorage, and lack of reinforcement continuity were identified as critical deficiencies contributing to the observed damage.

A recalibration of the model using a reduced behaviour factor (R = 4) for the 2007 code resulted in increased reinforcement demand in critical regions, confirming that the original design underestimated local demand, particularly at the base of the structure where seismic effects are most pronounced.

Parametric improvements, including an increase in wall thickness by 5 cm, demonstrated improved force distribution and a more uniform structural response, leading to reduced reinforcement demand and improved consistency of internal force transfer.

In addition, a base isolation system using soft lead rubber bearings was introduced as a seismic mitigation strategy. The results showed a significant reduction in internal forces and a more favourable distribution of interstory drifts, confirming the efficiency of isolation in reducing seismic demand and improving overall structural performance.

Overall, while the investigated tunnel-form system is conceptually efficient and well-suited for seismic regions, its real performance is highly dependent on construction quality and detailing practices. Improvements in boundary zone reinforcement, continuity of vertical reinforcement, rational wall strengthening, and the application of base isolation can significantly enhance the seismic safety and resilience of such structures.

REFERENCES

[1] Paul Somerville and Jacob Evans: Cause and surface faulting of the Türkiye earthquakes of February 6.2023.

[2] Abdullah Altindal & Aysegul Askan: Traditional seismic hazard analyses underestimate hazard levels when compared to observations from the 2023 Kahramanmaras earthquakes

[3] SAEE – Serbian Association for Earthquake Engineering

[4] TEC 2007: Turkish Earthquake Code 2007

[5] TBSC 2018: Turkish Building Seismic Code 2018

[6] EN 1992-1-1: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings. The European Committee for Standardization (CEN): Bruxelles, Belgium, 2015

[7] EN 1998-1: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings. The European Committee for Standardization (CEN): Bruxelles, Belgium, 2004.

[8] Srđan Janković: “Seismic calculation of reinforced concrete buildings in accordance with Eurocodes (with solved examples)