Yuzuncu Yil University
Journal of the Institute of Natural & Applied Sciences
https://dergipark.org.tr/en/pub/yyufbed

Research Article

Failure of Gedikbulak K-12 School Building on October 23, 2011, in Van, Türkiye Earthquake

Halit Cenan MERTOL

Atılım University, Engineering Faculty, Civil Engineering Department, 06830, Ankara, Türkiye
Halit Cenan MERTOL, ORCID No: 0000-0001-8058-5798
Corresponding author e-mail: cenan.mertol@atilim.edu.tr

Article Info

Received: 04.01.2023
Accepted: 31.03.2023
Online August 2023

DOI:10.53433/yyufbed.1229265

Keywords
10370 typical school building project,
Collapse,
Development and lap splice length,
Failure sequence

Abstract: A major earthquake with a magnitude M_w = 7.2 (M_L = 6.7) occurred in the Eastern part of Türkiye on October 23, 2011. The ground motion had a measured peak ground acceleration of 0.18×g. The earthquake damaged masonry and reinforced concrete structures ranging from minor cracking to total collapse. A 4-story reinforced concrete school building in Gedikbulak Village experienced total failure during this earthquake. This school building was a typical project (No: 10370) prepared by the Ministry of Education of Türkiye and this typical project was widely being used for the construction of school buildings in various locations throughout Türkiye. This paper explains the site observations of the Atılım University Reconnaissance Team carried out at the collapse site of the school building a few days after the main shock and detailed analysis with an emphasis on the reason for the failure of the school building. The analysis results indicated that the collapse happened due to the separation of the bottom of shear walls (in both x- and y-directions) from the foundation due to inadequate development and lap splice length of the plain reinforcing bars. Comments were made for existing school buildings to prevent any collapses in future earthquakes.

23 Ekim 2011 Van, Türkiye Depremi Sırasında Gedikbulak Okul Binasının Yıkılması

Makale Bilgileri

Geliş: 04.01.2023
Kabul: 31.03.2023
Online Ağustos 2023

DOI:10.53433/yyufbed.1229265

Anahtar Kelimeler
10370 tipik okul binası projesi,
Göçme,
Göçme adımları sıralaması,
Kenetlenme ve bindirme boyu

Öz: 23 Ekim 2011 tarihinde Türkiye'nin doğusunda M_w = 7,2 (M_L = 6,7) büyüklüğünde bir deprem meydana gelmiştir. Yer hareketinin en yüksek yer ivmesi 0.18×g olarak ölçülmüştür. Deprem, yığma ve betonarme yapılarda küçük çatlaklardan tamamen göçmeye kadar değişen hasarlara neden olmuştur. Bu depremde, Gedikbulak Köyü'ndeki 4 katlı betonarme okul binası tamamen yıkılmıştır. Bu okul binası Türkiye Cumhuriyeti Milli Eğitim Bakanlığı tarafından hazırlanan tipik bir projedir (No: 10370) ve bu tipik proje Türkiye'nin çeşitli yerlerinde okul binalarının yapımında yaygın olarak kullanılmıştır. Bu makalede, Atılım Üniversitesi Keşif Takımının ana şoktan birkaç gün sonra okul binasının yıkıldığı yerde yaptığı saha gözlemleri ve okul binasının yıkılma nedenleri üzerinde durularak detaylı analizleri anlatılmaktadır. Analiz sonuçları göçmenin, düz donatı çubuklarının yetersiz kenetlenme ve bindirme boyu nedeniyle perde duvarların tabanının (hem x- hem de y- yönlerinde) temelden ayrılmasından kaynaklandığını göstermiştir. Yapılan inceleme ve analizler sonucunda benzer mevcut okul binalarının gelecekteki depremlerde yıkılmaması için önerilerde bulunulmuştur.

1. Introduction

An earthquake with a magnitude of M_w = 7.2 (M_L = 6.7) occurred at 13:41 on October 23, 2011, in Van, Türkiye. This city had a population of 1,022,000 in 2011. The earthquake's epicenter was located approximately 30 km north of Van city center as shown in Figure 1. The measured peak ground acceleration (PGA) was equal to 0.18×g in the N-S direction. This station (Muradiye Station) was located 46 km northeast of the earthquake's epicenter. Details related to the ground motion data were provided by Baran et al. (2014). No closer measurement was obtained due to faulty equipment. The ground motion was felt within a 250 km radius of the epicenter. The officially declared life loss was 604 people and more than 4000 people were injured. The town of Ercis, located approximately 60 km north of Van city center, had the highest number of deaths and structural damages (Baran et al., 2014).

Figure 1. Location of the earthquake and photograph of school building before the earthquake.

Gedikbulak Village is located 47 km north of Van city center (between Van and Ercis) and 16 km northeast of the epicenter of the earthquake as shown in Figure 1. There were approximately 200 single or two story adobe masonry houses in the village. The only building made of reinforced concrete was the Gedikbulak K-12 School Building in the village. This school building was a typical project (No: 10370) prepared by the Ministry of Education of Türkiye and this typical project was widely being used for the construction of school buildings in various locations throughout Türkiye. Approximately 785 students were being educated in this building on a morning and afternoon session basis. The earthquake happened on a Sunday which prevented any life loss. However, there was a parents-teachers meeting on that day in the school which was luckily concluded one hour before the earthquake.

İnel et al. (2006) performed a study to evaluate the seismic performance of existing typical state buildings (hospitals and schools including 10370 typical school buildings) before the Van Earthquake in 2011. Based on the tests on concrete cores and Schmidt Hammer tests, the authors indicated that the state buildings constructed before 1998 had concrete strength ranging from 10 to 16 MPa and spacing of the transverse reinforcement ranging from 150 to 250 mm. Also, the authors observed that the steel reinforcement used in these buildings was S220 having a minimum yield strength of 220 MPa. The models of the buildings were subjected to pushover analysis, and the authors determined that the effect of poor concrete strength, low steel strength, and inadequate confinement were not significant in the seismic performance of the buildings having adequate shear walls.

Tapan et al. (2013) visited the damaged buildings after Van Earthquake. The authors also evaluated the Gedikbulak School. Based on on-site observations, it was concluded that the structural system looked adequate based on strength and stiffness subjected to moderate-level earthquakes. The collapse was associated with the deficiencies such as insufficient anchorage of the beam longitudinal reinforcement into the shear walls and inadequate lap splices of the longitudinal reinforcement of the shear walls. No detailed analysis was performed on the school building.

Damcı et al. (2015) presented their evaluations on the October 23, 2011, Van Earthquake. They also visited the Gedikbulak village and commented on the failure mechanisms based on their observations. It was concluded that faulty concrete production, defects in reinforcement details, and the poor quality of craft were the reasons for the failure of the regional buildings. Again, no detailed analysis was performed on the school building.

Orak & Celep (2017) performed a study to assess the seismic performance of the Gedikbulak K-12 School Building. The authors visited the collapse site after the earthquake. The building was modelled and analyzed using linear and nonlinear analysis methods. It was concluded that the reasons for the collapse of the building were poor material quality, poor grading of aggregates in concrete, and improper detailing related to the connections between shear walls, beams, and slabs. Similar deficiencies were observed at the beam-column connections by the authors. Detailed analysis was performed on the school building. The shortcoming of this evaluation was additional shear walls in the short direction. These shear walls were modelled in the analysis by mistake and did not appear in the collapsed (actual) school building.

This paper investigates the reason for the collapse of the Gedikbulak K-12 School Building on October 23, 2011, Earthquake, which was the only reinforced concrete building in Gedikbulak village. Atilim University Reconnaissance Team carried out a site visit to the village a few days after the earthquake. Detailed field observations were made. Numerous photographs were shot, and videos were recorded. Without endangering human life, activities such as measuring structural elements, determining reinforcement details, and evaluating concrete properties were performed on-site. After the site visit, plans of this typical school building project were obtained, and the building was modeled using SAP2000 Structural Analysis Software. The analysis results and field observations were evaluated and commented on to avoid any future collapse of such typical buildings in future earthquakes. This study is the only one that focuses on the detailed failure analysis of the Gedikbulak K-12 School Building after the 2011 Van Earthquake.

2. Structural System of the Building

The school building consisted of 3 stories. A photograph of the building before the earthquake is shown in Figure 1. The formwork and reinforcement plan drawings of the building were obtained from the Ministry of Education of Türkiye.

This school building was designed based on provisions specified by Specification for Structures to be Constructed in Disaster Areas (Ministry of Public Works and Settlement [MPWS], 1975). It can be assumed that the building was designed based on a horizontal earthquake load of 15% of the total weight of the structure based on this code. Turkish Earthquake Code was revised three times later, in 1998, 2007, and 2018. The latest version is called the Building Earthquake Specifications of Türkiye (MPWS, 2018).

The length of the building in the long direction (north-south direction) was 21.9 m, and the short direction (east-west direction) was 14.7 m. Center to center distances between the outer columns were 21.6 and 14.4 m in long and short directions, respectively. The plan area of the building per floor was 322 m^2, and the total area was 966 m^2. The height of each story was 3.2 m, and the total height of the building was 9.6 m. A typical plan view of the school building is shown in Figure 2.

The structural system of the building was a reinforced concrete frame having shear walls in both directions. The total areas of the shear walls strong in north-south (SW1 and SW2) and east-west directions (SW3 and SW4) were 5.25 and 2.50 m^2, respectively. The ratios of the area of the shear walls in north-south and east-west directions to the total area were 1.63% and 0.78%, respectively. Based on the drawings of the school building and site observations, the beams had 250×500, 250×700, 300×500, 300×700, and 400×700 mm, and the columns had 250×500, 300×500, and 400×500 mm dimensions. The thickness of the slab varied from 150 to 200 mm for typical floors and 200 mm for the whole roof floor.

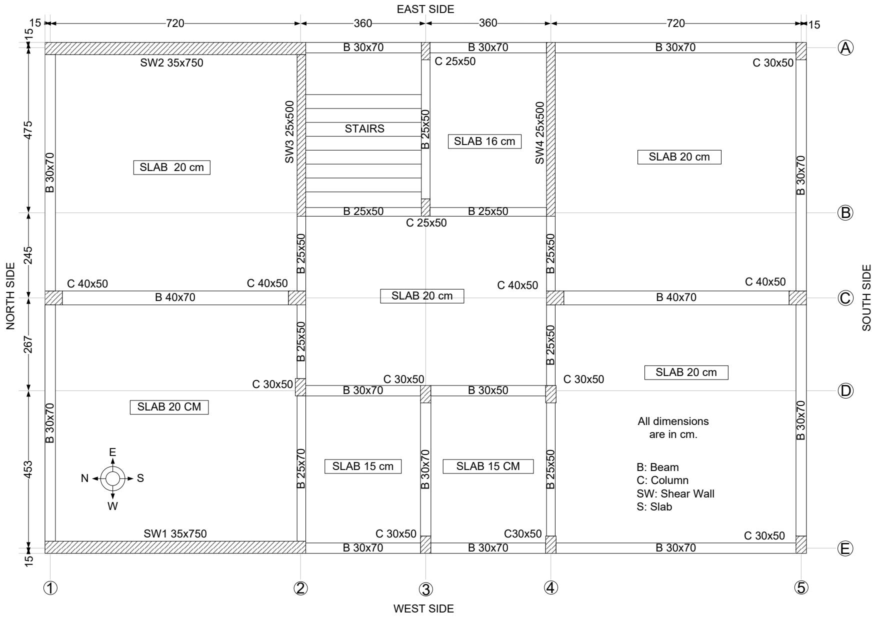

Figure 2. Typical plan view of the building.

3. Observed Behavior of the Building after the Earthquake

The side views of the building are shown in Figure 3 to Figure 5. The school building experienced total collapse. The locations between the 4 and 5 axes had collapsed, producing a sandwich-type failure of slabs on top of each other. The first floor had collapsed for the rest of the building, and the other floors were barely/partially standing with hefty damage. All the building has shifted towards the west direction, with fewer shear walls in the east-west direction compared to the north-south direction.

Figure 3. Southwest and west side views of the building.

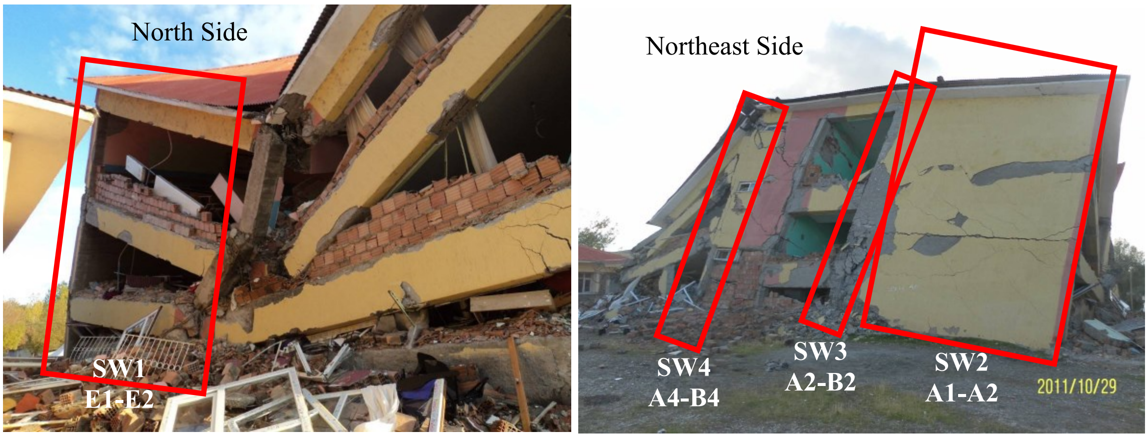

Figure 4. North and northeast side views of the building.

Figure 5. Southeast and south views of the building.

The base of the shear wall SW1 (SW1 E1-E2) had separated from the foundation due to the pull-out of the reinforcing bars and shifted approximately 1 m towards the west. This shear wall also experienced significant damage and bending in the weak direction towards the west, as shown in Figure 6. Shear wall SW2 (SW2 A1-A2) was significantly deformed in a weak direction. However, the base of this shear wall did not entirely separate from the foundation, as shown in Figure 4. However, a significant length of the longitudinal reinforcement was observed to pull-out, resulting in a detachment of approximately 100 mm between this shear wall SW2 and the foundation on one side. It was also observed that reinforcement bars were spliced at the base of the shear walls and columns (probable hinging locations) where the maximum load effects would occur.

Figure 6. Failure of shear wall SW1 (E1-E2 Axis).

The base of shear walls SW3 (SW3 A2-B2) and SW4 (SW4 A4-B4) were also separated from the foundation for the same reason and shifted approximately 0.5 m towards the north and west directions. Photographs of the motion of the bottom side of the shear wall SW4 are shown in Figure 7 and Figure 8.

Figure 7. Failure of shear wall SW4 (A4-B4).

Figure 8. Separation of shear wall SW4 from the foundation (A4-B4).

It was observed that the quality of the concrete used in the building was also poor. Although concrete compressive strength of 22.5 MPa was specified in the drawings and calculations, the in-situ concrete strength seemed much less. Large aggregates, improper placement, and insufficient concrete compaction were observed throughout the building.

Plain bars were used as longitudinal and transverse reinforcing steel in the construction of the building. Using deformed bars prior to Specification for Buildings to be Constructed in Earthquake Areas (MPWS, 2007) was not required as longitudinal reinforcement in column and beam elements in Türkiye. The specified yield strength of reinforcing steel was 220 MPa in the drawings and calculations. Poor detailing and craft were observed throughout the building, such as closely placed longitudinal reinforcement for beams and columns. The ends of the stirrups were not bent 135° to keep core concrete in place after the cover concrete was lost. Observations related to these problems are shown in Figure 9. Also, no adequate confinement of the stirrups (large spacing of transverse reinforcement) was observed at the ends of beams, end of columns, and beam-column joints (potential plastic hinge regions), as shown in Figure 9.

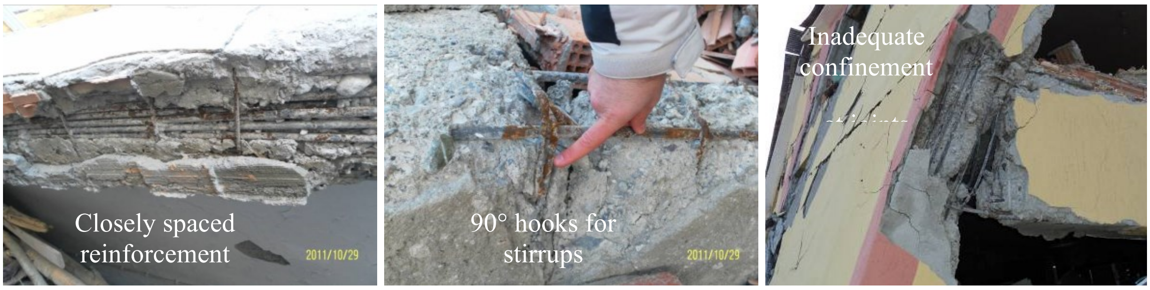

Figure 9. Problems related to reinforcement.

4. 3D Modeling

SAP2000 Structural Analysis Software was used to evaluate the behavior of the building subjected to seismic loading. The beams and columns were modeled using frame elements, whereas the shear walls and slabs were modeled using shell elements. Various views of the model are shown in Figure 10.

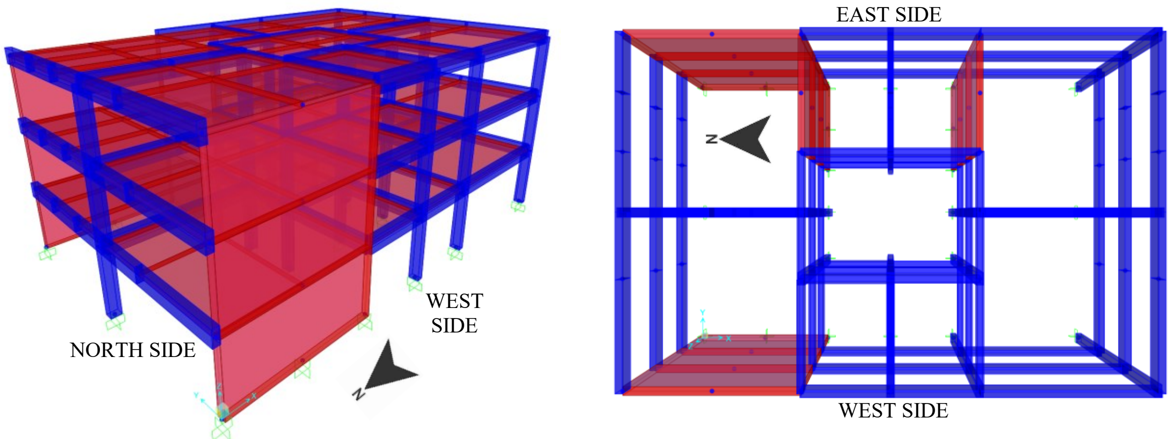

Figure 10. 3D views of the building model.

4.1. Materials

Although the specified concrete strength used in the calculations and drawings was 22.5 MPa, C16 concrete class was used in the analysis as requested by İnel et al. (2006). For the steel reinforcement, S220 was used. Values related to the material properties are shown in Table 1.

Table 1. Material properties used in the analysis

Definition	Value
C16 Characteristic Compressive Strength (MPa)=	16
C16 Modulus of Elasticity (MPa)=	27000
Unit Weight of Concrete (kNm^{-3})=	24
S220 Tensile Strength (MPa)=	220
S220 Modulus of Elasticity (MPa)=	200000
Unit Weight of Steel (kNm^{-3})=	77

4.2. Cross-sections and reinforcement configuration

The cross-sections defined in the model were determined based on the projects obtained from the Turkish Republic Ministry of Environment and Urbanization, Directorate General of Construction Works, and the observations at the site visit after the failure. Five different beam cross-sections, B250×500 mm, B250×700 mm, B300×500 mm, B300×700 mm, and B400×700 mm; three different columns cross-sections, C250×500 mm, C300×500 mm, and C400×500 mm; three different slabs, S150, S160, and S200 mm; and two different shear walls, SW1-SW2 350×7500 mm and SW3-SW4 250×5000 mm were used in the model. The dimensions and locations of beams, columns, slabs, and shear walls are shown in Figure 2. The reinforcement configurations of the beams and columns are shown in Figure 11. No confinement of stirrups was provided at the ends of columns and beams. For shear walls, ϕ10/35 cm reinforcement was used in both directions on both sides of the walls.

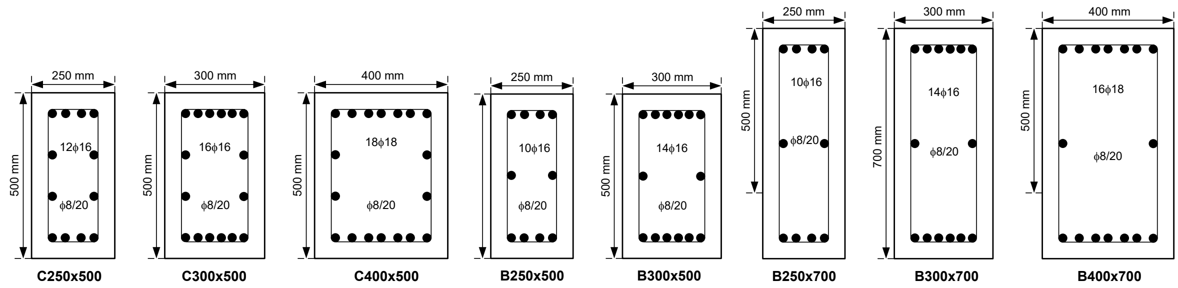

Figure 11. Reinforcement configuration for column and beams.

4.3. Loads

The loads applied on the building were determined based on the Turkish Standard 498 - Design Loads for Buildings (Turkish Standards Institute [TSE], 1997) and the observations at the site visit after the failure. The dead load (DL) due to the roof structure was taken as 1 kNm^{-2}. Additional DL on slabs due to plain concrete, slab cover, and ceiling plaster was estimated as 2 kNm^{-2}. The location of the building is a frigid region in Türkiye. Therefore, the plaster on the exterior walls was very thick. Based on on-site observations, this thickness was approximately equal to 50 mm. The walls consisted of hollow bricks (full brick for outer walls having 190 mm thickness and half brick for inner walls having 85 mm thickness) plaster on both sides (50+50 mm for exterior walls and 20+20 mm for interior walls). The estimated line DL due to walls on exterior beams was 10.94 kNm^{-1} [(0.19 m × 12.5 kNm^{-3} + 0.10 m × 20 kNm^{-3}) × (3.20 m - 0.70 m)] and on interior beams was 5.03 kNm^{-1} [(0.085 m × 12.5 kNm^{-3} + 0.04 m × 20 kNm^{-3}) × (3.20 m - 0.50 m)]. Half of the DL value of walls (5.47 kNm^{-1}) was used for the exterior walls having windows.

The model's live loads (LL) were estimated using TS498 (TSE, 1997). A LL value of 3.5 kNm^{-2} was used for the classrooms and general-purpose rooms, 5 kNm^{-2} was applied to the corridors, and 2 kNm^{-2} was assigned to the roof. During the earthquake, there was nobody in the school building.

Gedikbulak is located in the fourth snow region based on TS498 (TSE, 1997). The snow load for Region 4 up to 1000 m elevation is 1.6 kNm^{-2}. Since the elevation of Gedikbulak from sea level is 1742 m, the snow load is increased by 15% as required by TS498 (TSE, 1997). Therefore, the snow load is calculated as Pko = 1.6×1.15 = 1.84 kNm^{-2}. Snow load can be reduced if roof's slope is more significant than 30°. Since the roof of the building had a slope of less than 30°, the snow load was not reduced.

The earthquake loads were calculated using the Building Earthquake Specifications of Türkiye (MPWS, 2018). This code is very similar to the ASCE 7 (American Society of Civil Engineers [ASCE], 2016). Based on Building Importance Factor (I=1.5 for school buildings), Building Usage Class (BKS=1 for school buildings), Earthquake Design Class (DTS=1a for BKS=1 and SDS (site class modified spectral acceleration coefficient for a short period) ≥ 0.75), and Building Height Class (BYS=7 for DTS=1a and 7 m ≤ Building Height ≤ 10.5 m), the Building Target Performance Level is specified as Life Safety under DD2 Earthquake Ground Motion Level which is the motion having a mean recurrence interval of 475 years (10% probability of exceedance in 50 years). The corresponding Response Modification Coefficient (R) and Over-Strength Factor (D) were determined as 5 and 2.5, respectively, for a building with moment-resisting frames having limited-ductile columns and high-ductile shear walls without openings (Building System A22). Based on the site observations, it was assumed that the soil was soft clay, which a Site Class of ZE can represent. Using the Earthquake Hazard Map of Türkiye, the coefficient for a short period (S_s), the period at 1 s (S_1), and PGA were determined as 0.751, 0.183, and 0.315×g, respectively. After applying the soil modification coefficients (F_s=1.299, F_1=3.453), the design spectral acceleration factor for a short period (S_{DS}) and period at 1 s (S_{D1}) were calculated as 0.976 and 0.632, respectively. The corresponding T_A, T_B, and T_L were calculated as 0.130, 0.648, and 6 s, respectively. For the vertical effect of the earthquake motion, two-thirds of SDS×DL was used as specified by the code.

Another earthquake spectrum was constructed to simulate the earthquake that occurred in Gedikbulak Village on October 23, 2011. Since no closer data related to the ground motion was recorded, the measured PGA of 0.180×g was used to produce the spectrum. Corresponding S_s=0.41 and S_1=0.126 were read from the Earthquake Hazard Map of Türkiye. The same Response Modification Coefficient (R=5) and Over-Strength Factor (D=2.5) were used for this spectrum however, the Building Importance Factor was used as 1.0 to simulate the earthquake day behavior. Using the Soil Class Z_E, F_s and F_1 were determined as 1.952 and 3.966, respectively. Corresponding spectral acceleration coefficients and periods were calculated as S_{DS}=0.800, S_{D1}=0.500, T_A=0.125 s, T_B=0.624 s, and T_L=6 s. For the vertical effect of the earthquake motion, a similar approach (two-thirds of S_{DS}×DL) was used as the previous spectrum.

4.4. Analysis scenarios

The model was analyzed for four scenarios. The first scenario was produced to evaluate the behavior of this school building if it would resist the loads specified by the latest codes. The loads specified by TS498 (TSE, 1997) (DL, LL, and SL), the Horizontal Design Spectrum with a PGA of 0.315×g based on Building Earthquake Specifications of Türkiye (MPWS, 2018), and reinforced concrete design code TS500 (TSE, 2000) were used in the evaluation of the building. The practical section rigidity modifiers specified by Building Earthquake Specifications of Türkiye (MPWS, 2018) were also used. The supports of the bottom story columns of this model were assigned as fixed in this model. The parameters used in this model are shown in Table 2.

In the rest of the scenarios, the behavior of the building on the day of the collapse was modelled. In Model 2, the building was fixed at the supports, like in Model 1. Rollers were assigned for the supports under the shear walls SW3 and SW4 (shear walls in short direction) in Model 3 to simulate the motion of shear walls (separation from the foundation). In the last model (Model 4), all the supports of the shear walls were modeled as rollers. The DL was similar in all models. The LL for all the stories in the second, third, and fourth models were taken 1 kNm^{-2} (to simulate loads of the unfixed furniture, materials, equipment, etc., without LL of people) since at the time of the collapse, and there was nobody in the building. In addition, no SL was applied at the roof level on the models of the day of the collapse. The Horizontal Design Spectrum with a PGA of 0.180×g was used to calculate earthquake loads for these three models. Similar effective section rigidity modifiers were used in these models (Models 2, 3, and 4), except for the modifiers of shear walls to reflect the rigid behavior.

The equivalent seismic load method was used to calculate earthquake loads as specified by Building Earthquake Specifications of Türkiye (MPWS, 2018). The seismic weight of the building was calculated as DL+0.6×LL+0.3×SL. The combinations specified by Turkish Standard 500 - Requirements for Design and Construction of Reinforced Concrete Structures (TSE, 2000) and Building Earthquake Specifications of Türkiye (MPWS, 2018) were used in the analysis. The vertical load combination of 1.4×DL+1.6×LL was only used for Model 1. For Models 2, 3, and 4, only the summation of DL and LL was used as the vertical load combination. The earthquake combinations for all the models required considering 30% of orthogonal direction horizontal earthquake, 30% of the vertical earthquake, and 20% of snow loads in addition to 100% of DL, LL, and any direction earthquake load (e.g., 1.0×DL + 1.0×LL + 1.0×EQX + 0.3×EQY + 0.3×EQZ + 0.2×SL). Furthermore, the combinations also included the ±5% eccentricity of the mass center of the floor length in the perpendicular direction to the earthquake direction for both x- and y-directions. The calculated earthquake loads for each story using the building periods obtained from each SAP2000 model are shown in Table 2.

Table 2. Parameters used in analysis and results

Description	Model Based on Design Parameters	Model Based on the Day of Collapse		
	Fixed Support Conditions (Model 1)	**Fixed Support Conditions (Model 2)**	**Supports of SW3 and SW4 Roller, Others Fixed (Model 3)**	**Supports of Shear Walls Roller, Columns Fixed (Model 4)**
Total DL (kN)	11729			
Total LL (kN)	3013	933		
Total SL (kN)	572	0		
Seismic Weight of Each Story (DL+0.6×LL+ 0.3×SL) (kN)	Story 3: 3332+0.6×622+0.3×572=3877 Story 2: 4016+0.6×1195=4733 Story 1: 4016+0.6×1195=4733	Story 3: 3332+0.6×311=3518 Story 2: 4016+0.6×311=4203 Story 1: 4016+0.6×311=4203		
Total Seismic Weight (DL+0.6×LL+0.3×SL) (kN)	13343	11924		
Effective Section Rigidity Modifiers	**Shear Walls:** In Plane: Axial=0.5, Shear=0.5 **Shear Walls:** Out of Plane: Bending=0.25, Shear=1.0 **Slabs:** In Plane: Axial=0.25, Shear=0.25 **Slabs:** Out of Plane: Bending=0.25, Shear=1.0 **Beams:** Bending=0.35, Shear=1.0 **Beams:** Bending=0.70, Shear=1.0	**No reduction for shear walls.** **Slabs:** In Plane: Axial=0.25, Shear=0.25 **Slabs:** Out of Plane: Bending=0.25, Shear=1.0 **Beams:** Bending=0.35, Shear=1.0 **Beams:** Bending=0.70, Shear=1.0		
Building Factors:	Building Importance Factor (**I**) = **1.5** [Building Usage Class (**BKS**) = **1** for schools] Building Design Class (**DTS**) = **1a** (for BKS = 1 and $S_{DS} \geq 0.75$) Building Height Class (**BYS**) = **7** (for DTS = 1a and 7 ≤ Building Height ≤ 10.5 m) Target Performance: **Life Safety** under **DD-2** using **Strength Design** (for DTS = 1a and BYS > 3) Response Modification Coefficient (**R**) = **5** (Building System: A22) Over-strength Factor (**D**) = **2.5** (Building System: A22)	Building Importance Factor (**I**) = **1.0** Response Modification Coefficient (**R**) = **5** Over-strength Factor (**D**) = **2.5**		
Site Class	ZE			
How is PGA obtained?	Earthquake Hazard Map of Türkiye	Recorded PGA on site		
Spectrum Coefficients for DD-2 Earthquake Level	Site Coefficients: $S_s = 0.751$, $S_1 = 0.183$, **PGA = 0.315×g** Soil Coefficients: $F_s = 1.299$, $F_1 = 3.453$, Horizontal: $S_{DS} = 0.976$, $S_{D1} = 0.632$, $T_A = 0.130$ s, $T_B = 0.648$ s, $T_L = 6$ s Vertical: (2/3)×S_{DS}×DL	Site Coefficients: $S_s = 0.41$, $S_1 = 0.126$, **PGA = 0.180×g** Soil Coefficients: $F_s = 1.952$, $F_1 = 3.966$, Horizontal: $S_{DS} = 0.800$, $S_{D1} = 0.500$, $T_A = 0.125$ s, $T_B = 0.625$ s, $T_L = 6$ s Vertical: (2/3)×S_{DS}×DL		
Building Periods (s)	$T_x = 0.189$ $T_y = 0.323$	$T_x = 0.168$ $T_y = 0.230$	$T_x = 0.169$ $T_y = 0.413$	$T_x = 0.591$ $T_y = 0.478$
Structural System Behavior Factor	$R_x = 2.744$ $R_y = 2.915$	$R_x = 3.173$ $R_y = 3.421$	$R_x = 3.178$ $R_y = 4.152$	$R_x = 4.865$ $R_y = 4.416$
Spectrum Coefficients	$S(T_x) = 0.976$ $S(T_y) = 0.976$	$S(T_x)=0.800$ $S(T_y)=0.800$	$S(T_x)=0.800$ $S(T_y)=0.800$	$S(T_x)=0.800$ $S(T_y)=0.800$
Reduced Coefficients	$S(T_x)/R_x=0.356$ $S(T_y)/R_y=0.335$	$S(T_x)/R_x=0.252$ $S(T_y)/R_y=0.235$	$S(T_x)/R_x=0.252$ $S(T_y)/R_y=0.193$	$S(T_x)/R_x=0.165$ $S(T_y)/R_y=0.181$
Total Equivalent Seismic Load (Base Shear) (kN)	$F_x= 4745$ $F_y= 4465$	$F_x= 3008$ $F_y= 2790$	$F_x= 3003$ $F_y= 2298$	$F_x= 1962$ $F_y= 2161$
Story Earthquake Forces (kN)	$F_{3x}=2195$ $F_{2x}=1700$ $F_{1x}=850$ $F_{3y}=2066$ $F_{2y}=1600$ $F_{1y}=800$ $F_{3z}=2167$ $F_{2z}=2612$ $F_{1z}=2612$	$F_{3x}=1407$ $F_{2x}=1067$ $F_{1x}=533$ $F_{3y}=1305$ $F_{2y}=990$ $F_{1y}=495$ $F_{3z}=1778$ $F_{2z}=2143$ $F_{1z}=2143$	$F_{3x}=1405$ $F_{2x}=1065$ $F_{1x}=533$ $F_{3y}=1075$ $F_{2y}=815$ $F_{1y}=408$ $F_{3z}=1778$ $F_{2z}=2143$ $F_{1z}=2143$	$F_{3x}=918$ $F_{2x}=696$ $F_{1x}=348$ $F_{3y}=1011$ $F_{2y}=767$ $F_{1y}=383$ $F_{3z}=1778$ $F_{2z}=2143$ $F_{1z}=2143$

5. Results and Discussions

5.1. Elastic analysis

The models were linearly analyzed using the equivalent seismic load method for four conditions. A total of 25 combinations were used in the analysis, including the ±5% accidental eccentricity in both x- and y-directions. The vertical effect of the earthquake in the z-direction was also considered. The primary mode of failure for the first three models was in the y-direction. For the last model, where all the supports of the shear walls were rollers, the primary mode of failure was torsional. After the analysis, structural members of the building were checked against bending, shear, and axial load. The first model used material factors γ_{mc}=1.5 and γ_{ms}=1.15 for concrete and steel strengths, respectively. No material factors were used to evaluate Models 2, 3, and 4. The results of the capacity check considering axial, shear, and flexure are shown in Figure 12. These results are tabulated in Table 3. The red members in these figures have capacity ratios of more than 1.0, meaning insufficient members.

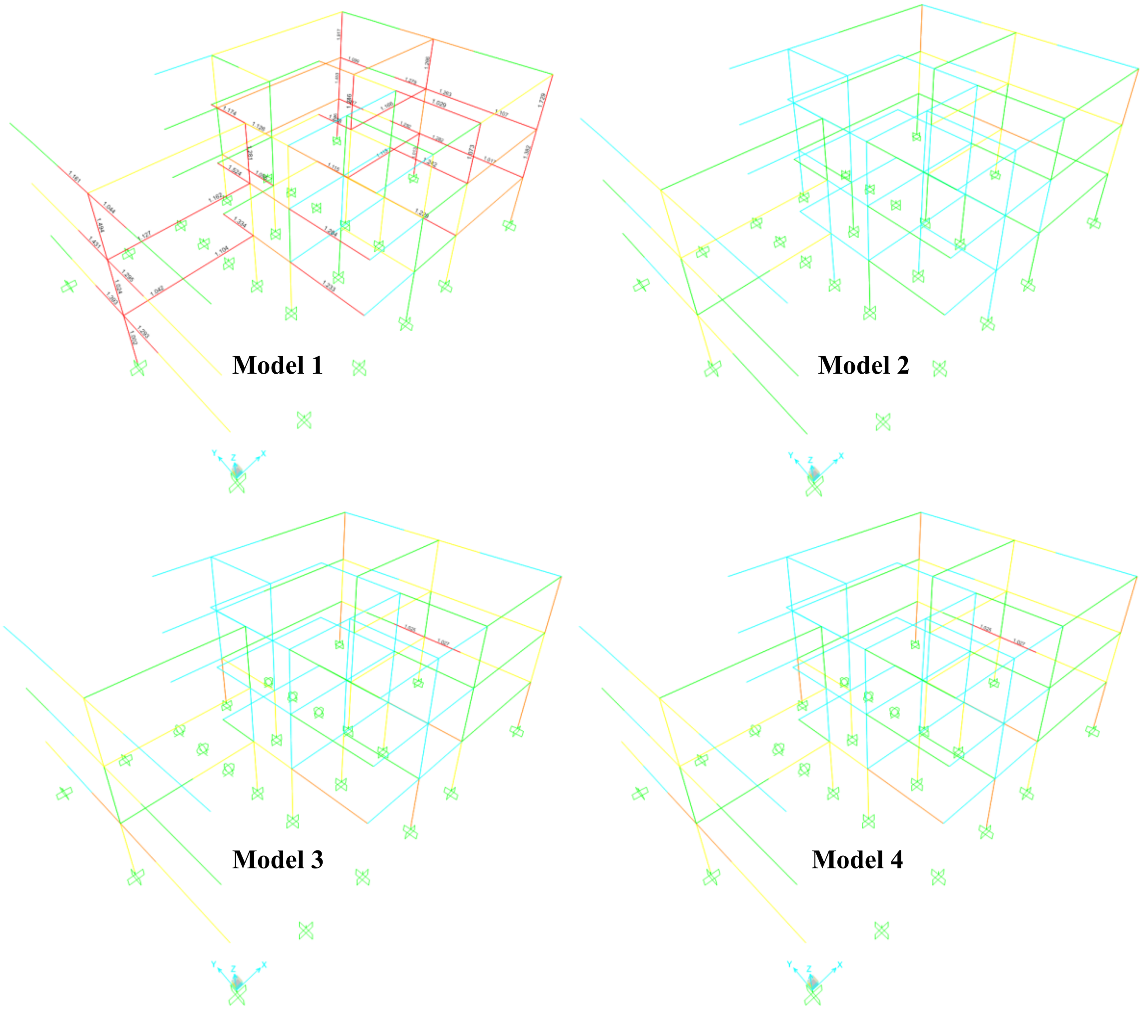

Figure 12. Results of capacity check of models (only the ratios more than 1.0 are shown).

Table 3. Tabulated analysis results of all models

Description	**Model Based on PGA=0.315×g**	**Model Based on the Day of Collapse (PGA=0.180×g)**		
	Fixed Support Conditions (Model 1)	**Fixed Support Conditions (Model 2)**	**Supports of SW3 and SW4 Roller, Others Fixed (Model 3)**	**Supports of Shear Walls Roller, Columns Fixed (Model 4)**
Insufficient Elements	13 columns 26 beams	0 columns 0 beams	0 columns 2 beams	17 columns 10 beams
Maximum Capacity Ratio	1.917 for columns 1.524 for beams	Less than 1 for all columns and beams	Less than 1 for columns 1.027 for beams	3.291 for columns 1.328 for beams
Maximum Displacement (mm)	-	U_1=1.25 U_2=3.57 U_3=0.92	U_1=2.00 U_2=10.86 U_3=0.99	U_1=20.71 U_2=28.78 U_3=15.43

The results of the elastic analysis of Model 1 indicate that the building would not resist the design loads specified by current codes since the capacity ratio of some of the members exceeded the limit of 1.0. Note that if this building were designed using current codes, more robust materials (C25 and S420 shall be used as a minimum) and larger cross-sections would be used to avoid insufficient members in the elastic analysis.

Results obtained from the elastic analysis of Model 2 indicate that the school building should have survived the earthquake that occurred on October 23, 2011. No beams or columns exceeded the capacity ratio of 1.0. In addition, the levels of load effects resisted by the shear walls were all under acceptable limits. However, the building collapsed totally. From the behavior of Model 2, it can be concluded that the failure of this building on the day of the earthquake was not related to the capacity exceedance (moment, shear, or axial load) of the structural members.

The reason for the collapse can be explained based on the results of the analysis of Models 3 and 4. Initially (as indicated by Model 3), the supports of the shear walls oriented in a short direction (SW3 and SW4) were separated from the foundation due to inadequate development and lap splice length of the reinforcing bars. Note that the reinforcing bars used in the building were all plain bars without any ribs. The poor bonding characteristics of plain reinforcing bars with low concrete quality resulted in pulling out of these reinforcing bars out of concrete elements. Later, this separation continued (as indicated by Model 4) with shear walls oriented in a long direction (SW1 and SW2). The loss of the connections of shear walls to the foundation resulted from excessive building motion in both directions. At this point, the moment capacity of 17 columns was exceeded with a maximum capacity ratio exceedance of more than three times, resulting in a total collapse of the building.

For the existing school buildings with the same typical projects (No: 10370), the bottom parts of the shear walls shall be strengthened to prevent any pull-out of the reinforcing bars resulting in the separation of the shear walls from the foundation. Later, the insufficient members (both beams and columns) of Model 1 shall be strengthened to resist the design loads calculated based on the current code practices. Additional shear walls in the short direction may be constructed as a final touch to have similar ratios of the areas of shear walls in both directions.

5.2. Nonlinear pushover analysis

A nonlinear pushover analysis was performed for the four models explained above to generate the overall force-deformation response and identify key performance levels in force and deformation. In these models, hinge locations are defined at the ends of the beams and columns of the building. Auto hinge property (P-M2-M3) specified in Table 10-8 of ASCE 41 (2013) (readily available in SAP2000) for the columns and auto hinge property (M3) specified in Table 10-7 of ASCE 41 (2013) (readily available in SAP2000) for the beams were used at the ends of the columns and beams located at 5% relative distance from both ends. Also, using the frame hinge assignment overwrites, the line objects at hinges were defined to be automatically subdivided into 2% of the relative length of the line elements at hinges. The start of the nonlinear pushover analysis of the structure started was continued from the state at the end of the nonlinear DL Case. The displacement-controlled load was applied separately as an acceleration at a roof-level joint in both x- and y-directions. Resultant base shear vs. monitored displacement relationships were obtained at the end of the analysis, as shown in Figure 13. These graphs show that the structure had greater stiffness in the x-direction than in the y-direction, which was evident due to the number of shear walls in different directions. Therefore, the structure's failure mode in the direction West (y-direction in the models) can be defined as the dominant mode of failure, and evaluations related to this direction will be explained in detail afterward. The capacity of Model 4 was the lowest in both directions.

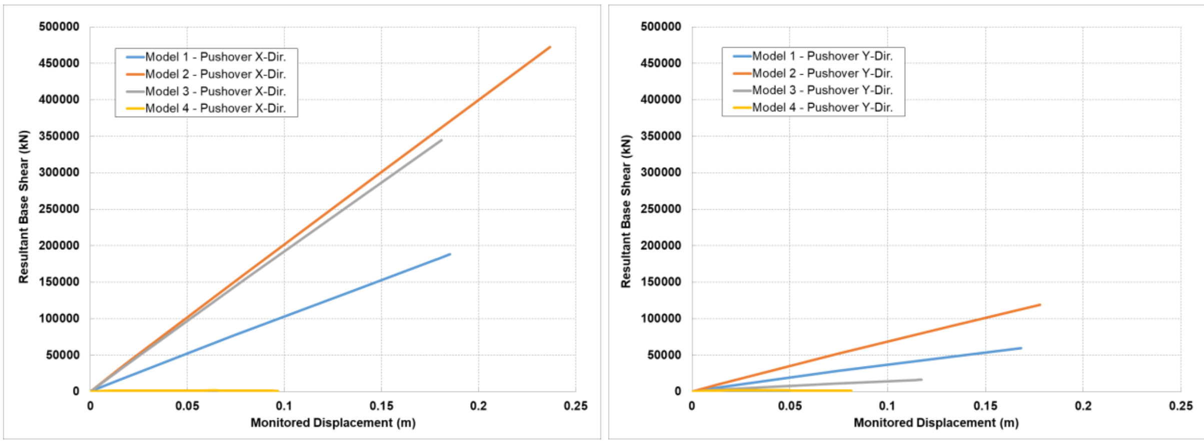

Figure 13. Resultant base shear vs. monitored displacement relationships in x- and x-directions.

The analysis results were compared to Capacity Spectrum Method (CSM) defined in ATC 40 (Applied Technology Council [ATC], 1996). The coefficients used to produce the 5% damped elastic response spectrums are shown in Table 4. The results of this comparison and the data related to the performance point of the models are shown in Table 5 and Figure 14. Based on the table, the values for the effective period, spectral acceleration, spectral displacement, base shear, and roof displacement of Model 1 at the performance point in the y-direction were equal to 0.303 s, 0.926 ms^{-2}, 0.0022 m, 12489 kN, and 0.032 m, respectively. In spectral acceleration vs. displacement graphs (Figure 14), the capacity and demand curves are plotted in green and orange colors, respectively. These figures indicate that Models 1, 2, and 3 had capacities beyond the demands implying that they would survive the specified earthquakes. However, Model 4 had a capacity significantly below the demand curve, which indicates that the model would fail under the specified loads and conditions without reaching a performance point.

Table 4. Coefficients of the 5% damped elastic response spectrums (CSM)

Description	**Model Based on PGA=0.315×g**	**Model Based on the Day of Collapse (PGA=0.180×g)**		
	Fixed Support Conditions (Model 1)	**Fixed Support Conditions (Model 2)**	**Supports of SW3 and SW4 Roller, Others Fixed (Model 3)**	**Supports of Shear Walls Roller, Columns Fixed (Model 4)**
C_A (=S_{DS}/2.5)	0.3904		0.32	
C_V (=S_{D1})	0.632		0.5	
T_A (=T_A) (s)	0.130		0.125	
T_S (=T_B) (s)	0.648		0.625	

Table 5. Results at performance point in the y-direction

Description	**Model Based on PGA=0.315×g**	**Model Based on the Day of Collapse (PGA=0.180×g)**		
	Fixed Support Conditions (Model 1)	**Fixed Support Conditions (Model 2)**	**Supports of SW3 and SW4 Roller, Others Fixed (Model 3)**	**Supports of Shear Walls Roller, Columns Fixed (Model 4)**
T_{eff} (s)	0.303	0.211	0.484	NA
S_a (ms^{-2})	0.926	0.8	0.598	NA
PUSH-Y S_d (m)	0.022	0.0089	0.041	NA
V (kN)	12489	9659	7446	NA
D (m)	0.032	0.013	0.046	NA
Step Numbers Performance Point Lies in between	1 & 2	1 & 2	1 & 2	NA

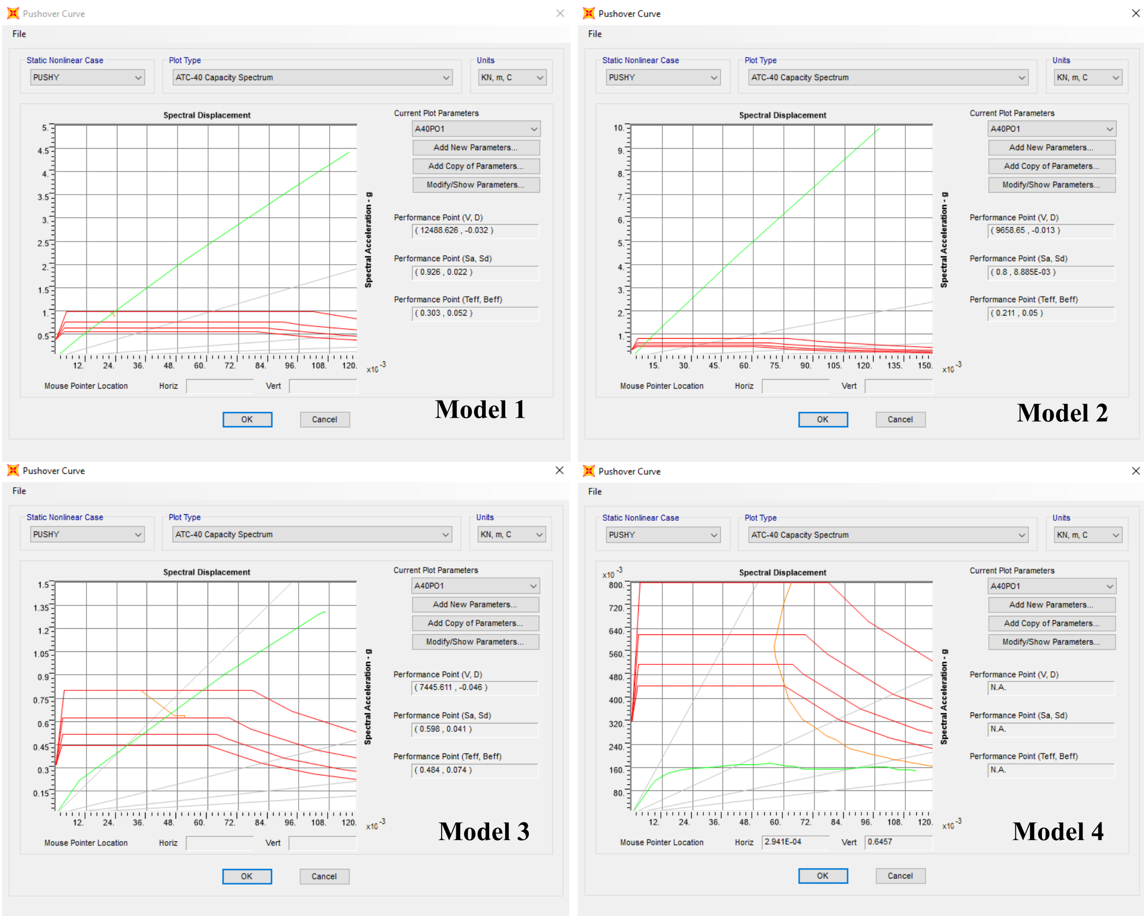

Figure 14. Capacity and ATC-40 Demand comparisons for y-direction pushover analysis.

The structure's hinge evaluation is performed using the graph shown in Figure 15 (ATC 40, 1996). In this figure point A refers to the original undeformed state of the hinge. Elastic deformation occurs between points A and B. At point B, the hinge starts yielding. Three performance levels, namely, immediate occupancy (IO), life safety (LS), and collapse prevention (CP), are defined between points B and C. After the event, the hinge shall be safe and usable at the IO performance level. For the LS performance level, the hinge may be damaged, which will not provide any threat to life safety. Although the hinge is severely damaged in the collapse prevention performance level, the collapse of the hinge is prevented. The ultimate capacity of the hinge is reached at point C. Point D represents the residual strength of the hinge. After this point collapse of the collapse is initialized, and the total failure of the hinge happens at point E.

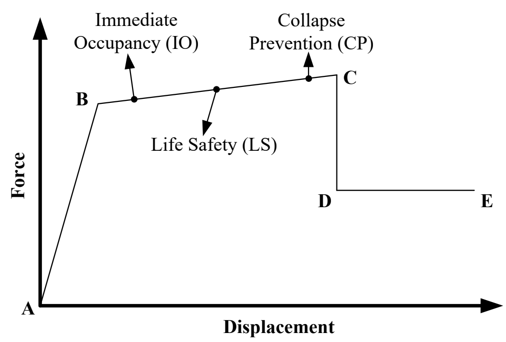

Figure 15. Force-displacement relationship of hinges.

The SAP2000 Software provided step numbers of the pushover analysis where the performance points are located, as shown in Table 5. Performance levels of hinges of all the models at various steps of pushover analysis in y-direction are shown in Figure 16. Based on this figure, the hinges of Model 1 and 2 experienced deformations at the yielding level. This level was located before the IO level resulting in no or minimal damage to the structure at those levels of ground motions. Some hinges of Model 3 would reach the IO level, producing no failure at that level of ground motion. However, Model 4 had failed hinges at the maximum analyzable state (Step 41) using SAP2000 and still did not reach a performance point. This confirms the findings of the elastic analysis in which the failure of the structure is associated with the loss of the connections of shear walls to the foundation resulting in excessive motion of the building in both directions.

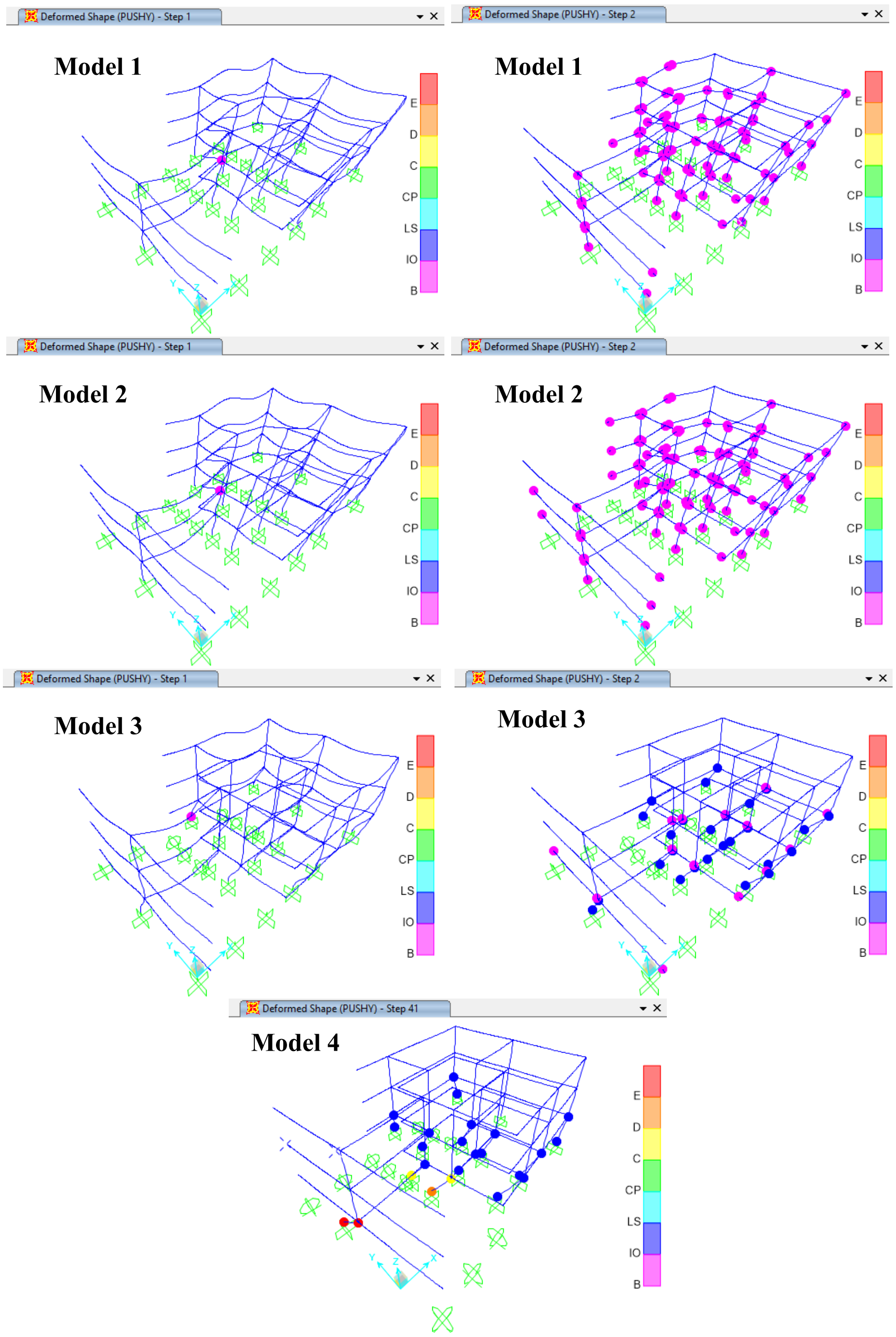

Figure 16. Performance levels of hinges at various steps of pushover analysis in y-direction.

6. Summary and Conclusions

The Gedikbulak K-12 School Building, which was a typical project widely being constructed in various locations throughout Türkiye, collapsed in the October 23, 2011 Van Earthquake. Site visit to the school building was performed a few days after the earthquake. Plans of this typical school building project were obtained and detailed elastic and nonlinear analyses were performed on the constructed models using SAP2000 Structural Analysis Software. Following conclusions were drawn:

- If the integrity of the shear walls and foundation were ensured (Model 2), the school building should have survived the earthquake occurred on October 23, 2011 based on both linear elastic and nonlinear pushover analyses.
- The collapse happened due to the separation of the bottom of shear walls (in both x- and y-directions) from the foundation due to inadequate development and lap splice length of the plain reinforcing bars. This was verified using both linear elastic and nonlinear pushover analyses (Model 4). The performance analysis would not capture these vulnerabilities if not done in lieu of the observations in Gedikbulak Village.
- For the existing school buildings with the same typical projects, the bottom parts of the shear walls shall be strengthened to prevent any pull out of the reinforcing bars. The insufficient members shall be strengthened to resist the design loads calculated based on the current code practices, and additional shear walls shall be added to the system to have similar ratios of the areas of shear walls in both directions.

Acknowledgments

The precious efforts of the members of the team, Prof. Dr. Eray Baran and Mr. Ferit Yılmaz (you will be remembered in our hearts), are very much appreciated. The local contacts in Van, Şemsettin Bakır, Necdet Telsaç, Kamuran Turgut, and Ercan Altunlı are also acknowledged.

References

American Society of Civil Engineers (ASCE). (2013). *Seismic evaluation and retrofit of existing buildings*: ASCE Standard 41-13.

American Society of Civil Engineers (ASCE). (2016). *Minimum design loads for buildings and other structures*: ASCE/SEI Standard 7-16.

Applied Technology Council (ATC). (1996). ATC 40: *Seismic evaluation and retrofit of concrete buildings*.

Baran, E., Mertol, H. C., & Gunes, B. (2014). Damage in reinforced concrete buildings during October 23 and November 9, 2011 Van, Turkey earthquakes. *Journal of Performance of Constructed Facilities,* 28(3), 466-479. doi:10.1061/(ASCE)CF.1943-5509.0000396

Damcı, E., Temur, R., Bekdaş, G., & Sayın, B. (2015). Damages and causes on the structures during the October 23, 2011 Van earthquake in Turkey. *Case Studies in Construction Materials,* 3, 112-131. doi:10.1016/j.cscm.2015.10.001

İnel, M., Bilgin, H., & Özmen, H. B. (2006). Mevcut kamu yapılarının performans değerlendirmesi [Performance evaluation of existing state buildings] [In Turkish]. *IMO Turkey Engineering News*, 444-445, 64-71.

Ministry of Public Works and Settlement (MPWS). (1975). *Specification for structures to be constructed in disaster areas* [In Turkish]. Ankara, Türkiye.

Ministry of Public Works and Settlement (MPWS). (1998). *Specification for structures to be constructed in disaster areas* [In Turkish]. Ankara, Türkiye.

Ministry of Public Works and Settlement (MPWS). (2007). *Specification for structures to be constructed in disaster areas* [In Turkish]. Ankara, Türkiye.

Ministry of Public Works and Settlement (MPWS). (2018). *Building earthquake specifications of Türkiye* [In Turkish]. Ankara, Türkiye.

Orak, M. S., & Celep, Z. (2017). Gedikbulak okul binasının sismik performansı hakkında [Seismic performance of Gedikbulak school building revisited] [In Turkish]. *IMO Teknik Dergi*, Technical Note, 28(2), 7889-7896. doi:10.18400/tekderg.304103

Tapan, M., Cömert, M., Demir, C., Sayan, Y., Orakçal, K., & İlki, A. (2013). Failures of structures during the October 23, 2011 Tabanlı (Van) and November 9, 2011 Edremit (Van) earthquakes in Turkey. *Engineering Failure Analysis,* 34, 606-628. doi:10.1016/j.engfailanal.2013.02.013

Turkish Standards Institute (TSE). (1997). TS498 *Turkish Standard - Design loads for buildings* [In Turkish]. Ankara, Türkiye

Turkish Standards Institute (TSE). (2000). TS500 *Requirements for design and construction of reinforced concrete structures* [In Turkish]. Ankara, Türkiye.