Annex H

Geology and Soils

H1 INTRODUCTION

This Annex presents the methodology, findings and recommendations of the geology and soils impact assessment of the Gaziantep Integrated Healthcare Campus (the Project), located in the Şahinbey District of Gaziantep, southeast Turkey. The assessment considers Project activities during construction and operation with the potential to cause impacts to geology and soils.

H1.1 SCOPE OF THE GEOLOGY AND SOILS ASSESSMENT

This Annex presents an evaluation of the Project site conditions in relation to geological and geophysical characteristics as well as seismic risks, soil conditions and the potential for contaminated land. The criteria used to assess impact significance are provided, followed by a description of the baseline situation. Potential significant impacts are then discussed and the proposed mitigation measures presented.

H1.2 STUDY AREA

The study area for this assessment covered the immediate Project Site and the Gaziantep Province.

H2 METHODOLOGY

H2.1 RELEVANT DOCUMENTS, STANDARDS AND GUIDELINES

The following Turkish regulations are relevant for this Project. They regulate the construction of hospital buildings in seismic zones, soil pollution and point-source contamination:

- Regulation on Buildings to be built in Seismic Zones (Official Gazette date/no: 06.03.2007/26454); and

- Regulation on Soil Pollution Control and Point-Source Contaminated Sites ('RSPC') (Official Gazette Date/Number: 08.06.2010/ 27605).

There is no specific EU framework directive related to soil pollution control.

H2.2 DESKTOP ANALYSIS

A detailed desktop analysis was undertaken to collect information on the baseline geology and soil conditions. This included obtaining information from relevant authorities including the General Directorate of Mineral Research and Exploration (MTA) and the General Directorate of Disaster Affairs (AFAD). In addition, the sources listed below were used to gather information on baseline conditions:

- The General Directorate of Mineral Research and Exploration (MTA) publications, (http://www.mta.gov.tr);

- Gaziantep Environmental Status Report (2014);

- Ministry of Public Works and Settlement, General Directorate of Disaster Affairs (AFAD) official web site (http://www.gaziantepafad.gov.tr);

- Spatial and Statistical Distribution of Natural Disasters in Turkey report prepared by Disaster Affairs General Directorate of the Ministry of Public Works and Settlement;

- Soil Investigation Report (including Geophysical Survey and Seismic Hazard (Risk) Analysis Study) prepared by PLATO Underground Research Engineering Inc. for the Gaziantep IHC Project;

- Drilling-based Ground and Sub-surface Survey Report (including Geophysical Survey and Seismic Hazard (Risk) Analysis Study) prepared by Enar Engineering, Architecture and Consultancy Inc. for the Gaziantep IHC Project (March 2016). This report has been prepared based on the findings of Soil Investigation Study prepared by PLATO Underground Research Engineering Inc. for the Gaziantep IHC study; and

- Scientific papers (detailed references are provided where relevant).

H2.3 *FIELD SURVEY*

Field surveys included a Soil Investigation Study, Geophysical Survey and a Seismic Hazard (risk) Analysis Study, which were all conducted by PLATO Underground Research Engineering Inc.

The Soil Investigation Study was conducted between 27/12/2014 and 20/08/2015 to obtain geological information and determine geotechnical parameters for the Project Site. Foundation boreholes were drilled to depths of between 21 m and 31 m at 76 locations in the area. Drilling of 12 additional foundation borehole locations was performed by PLATO Underground Research Engineering Inc., which was completed in February 2016. During drilling works on site, five core samples were taken at varying depths within each hole and sent for laboratory analysis.

The Geophysical Survey was carried out on the Project Site on 26/05/2015. As part of this study, 30 seismic profile locations were selected. Seismic records were taken with the Multi-channel Analysis of Surface Waves (MASW) method in order to obtain 1-D seismic wave velocity logs along the planned seismic profiles.

Moreover, a Seismic Hazard (Risk) Analysis Study was conducted to determine the likely seismic hazard (risk) at the Project Site how this should feed into the design of the Project.

Based on the findings of Soil Investigation Study prepared by PLATO Underground Research Engineering Inc. for the Gaziantep IHC study, a Drilling-based Ground and Sub-surface Survey Study has been conducted by Enar Underground Research Engineering, Architecture and Consultancy Inc.

H2.4 *IMPACT ASSESSMENT METHODOLOGY*

The assessment of likely impacts is determined by assigning ratings for impact magnitude and the sensitivity/vulnerability/importance of receptors/ resources as described in *Volume I, Chapter 5*. Once the magnitude of the impact and sensitivity of the resource/receptor is characterised, impact significance is assigned using the significance matrix presented in *Volume I, Chapter 5*.

Table H2.1 and *Table H2.2* describe the designations used for impact magnitude and resource sensitivity/vulnerability/importance when assessing impacts to geology and soils.

Table H2.1 Magnitude of Impact on Geology and Soils

Magnitude	Definition
Large	• Continuous/long-term oil spills during construction activities on soils and during operation (e.g. accidents) (concentrations of pollutants in the soil defined in the Soil Pollution Control Regulations are exceeded to cause long term cancer and hazard risk). • In case of disturbance of contaminated soils, increase contamination in nearby non-contaminated soils to above the background level that will be hazard to human health. • Major impacts on the integrity of structures and functionality of the Project (e.g. collapse of the buildings) during a seismic event.
Medium	• Continuous/long-term oil spills during construction activities on soils and during operation (e.g. accidents) (concentrations of pollutants in the soil defined in the Soil Pollution Control Regulations are exceeded above the generic contamination levels but below the long term cancer and hazard risk). • In case of disturbance of existing contaminated soils: increase contamination in nearby non-contaminated soils to above the background level that are above the generic risk levels stated in the Soil Pollution Control Regulations but below long term cancer and hazard. • Moderate impacts on the integrity of structures and functionality of the Project (e.g. major cracks in the structures) during a seismic event.
Small	• Temporary small-scale oil spills during construction and during operation (e.g. accidents) activities on soils that lead to contamination below generic contamination levels stated in the Turkish Regulation on Soil Pollution Control and Point Source Contaminated Sites (Soil Pollution Control Regulations). • In case of disturbance of existing contaminated soils: increase contamination in nearby non-contaminated soils to above the background level but below the generic contamination levels stated in the Soil Pollution Control Regulations. • Minor impacts on the integrity of structures and functionality of the Project (e.g. minor cracks in the structures) during a seismic event.
Negligible	• Temporary use of land (with soil surface) for the storage of excavated materials and construction equipment with no or little impact that is recoverable within a short time scale. • No earthquake impacts.

Table H2.2 Geology and Soils Resource Sensitivity/Vulnerability/Importance

Value	Definition
Low	• Soils that are not used for agricultural purposes • Areas of no geological importance Areas falling into 5th degree seismic zone (as detailed by AFAD [1])
Medium	• Soils with good quality to support agricultural production • Geological site of local/regional importance Areas falling into 3rd – 4th degree seismic zones (as detailed by AFAD [2])
High	• Highly fertile soils for agricultural production • Geological site of high importance Areas falling into 1st – 2nd degree seismic zones (as detailed by AFAD [3])

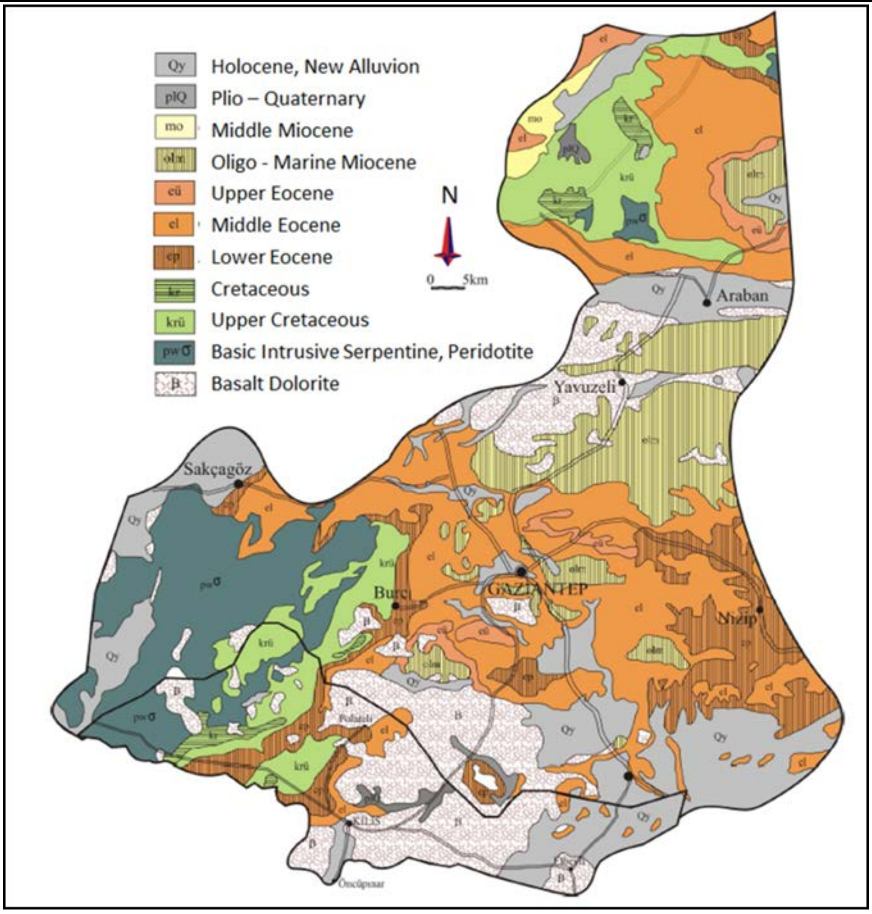
(1) http://www.deprem.gov.tr/en/Category/earthquake-zoning-map-96531.
(2) http://www.deprem.gov.tr/en/Category/earthquake-zoning-map-96531.
(3) http://www.deprem.gov.tr/en/Category/earthquake-zoning-map-96531.

H3 BASELINE

H3.1 REGIONAL GEOLOGY

Late Cretaceous and Miocene collisions of the Arabian, Anatolian and Eurasian plates, created the conditions for the formation of the surface and subsurface structures in the Gaziantep Basin. The structural evolution of the foreland area was influenced by the late Cretaceous (Maastrichtian) emplacement of the Kocali–Karadut ophiolite complex, which induced subsidence in the north western zone of the Kastel Basin during the early Alpine Orogeny. The Dead Sea Fault originated in the Red Sea in Miocene time and it propagated towards the northwest in the Suez Gulf and the north-northeast in southeast Turkey to influence the structural evolution of the Gaziantep Basin. These two major tectonic events produced many thrusts, thrust-related subsurface and surface anticlines, faults, fractures, flower structures and basaltic flows in the area (Coskun and Coskun, 2000 [1]). The Geological map of Gaziantep Province is shown in *Figure H3.1.*

Figure H3.1 Geological Map of Gaziantep Province

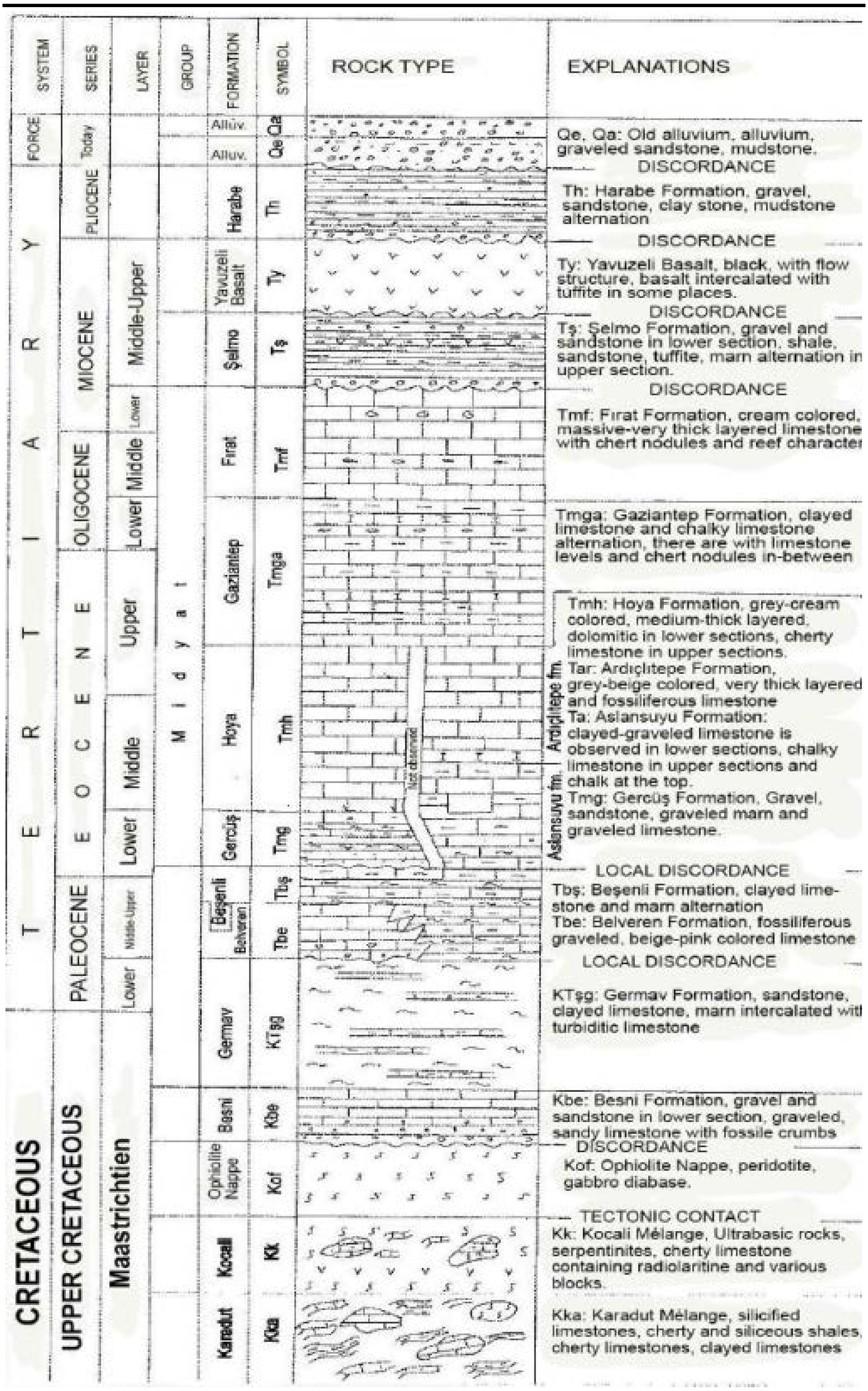


Source: The General Directorate of Mineral Research and Exploration (MTA), http://www.mta.gov.tr/

(1) Coskun B. and Coskun B. (2000). The Dead Sea Fault and related subsurface structures, Gaziantep Basin, southeast Turkey. Geol. Mag. 137 (2), 175 – 192

Geological studies by the MTA were taken as the basis for the survey area and its surroundings during the Soil Investigation Study. The 1/100,000-scaled geological maps and reports compiled during these studies were used to list the array of geological formations from bottom to the top. The generalised stratigraphic column of the Gaziantep Province is shown in *Figure H3.2* and a detailed characterisation of the formations is summarised below.

Figure H3.2 Gaziantep K24 Plate Generalised Vertical Section



Source: Soil Investigation Report prepared by PLATO Underground Research Engineering Inc. for the Gaziantep IHC Project (August 2015)

H3.1.1 Aslansuyu Formation (TA)

The unit consists of an argillaceous-gravel limestone and chalk (Güvenc (1973)). The formation begins with argillaceous-gravel limestone. This limestone unit is grey-beige in color, medium-thick layered, cherty and gravel contents. There is chalky limestone on top of this layer which consists of yellow-black chert strips with beige-whitish grey color, medium-thick layers and abundant micro-fauna. Intermediate layers of argillaceous limestone, green-grey claystone and very fine-grained sandstone are observed in some places between these layers. The unit – the uppermost level of which consists of white, thick and bad layered, loose-textured chalk – was settled in micro-facies environment "*on the edge of the basin or on the edge of the deep shelf*." The thickness of the formation is 500 m in the Hayırlı Stream and varies between 500 – 300 m in the map area. There is contact with the overlying Ardıçlıtepe Formation. The formation dates to the Lower-Middle Eocene.

H3.1.2 Ardiclitepe Formation (Tar)

The formation consists of limestone in general. The unit starts with elastic limestone and chalky limestone alternation at the bottom. Limestone is thick-very thick layered; chalky limestone is whitish-pale yellow-beige in color and has medium-thick layers. Towards the upper side of the layer, the unit consists of cherty limestone in lens and knob form, pale yellow-grey-beige in color, with thick-very thick layers and without any layers in some places, hard-steady, porous, with melting gaps, with micro and macro-fauna and yellow-brown-black in color. The thickness of the formation generally varies between 50 – 200 m and has a gradual transitive contact on Aslansuyu Formation. There is contact with the overlyingGaziantep Formation. The formation dates back to Middle Eocene (Upper Lutetian) – Upper Eocene (Priabonian).

H3.1.3 Gaziantep Formation (Tmga)

This unit consists of argillaceous limestone, limestone and chalk as defined and named by Wilson and Krummenacher (1957). The surface of the formation is in the form of argillaceous limestone, limestone and chalky limestone in soft topography. In some places, there is thick-layered limestone instead of this argillaceous and chalky limestone. The argillaceous limestone is whitish grey-cream-off-yellow in color, with thin-medium layers, grained structure and with algae and coral in some places. Argillaceous limestone and chalky limestone is settled in micro-facies environment "*on the edge of the basin or on the edge of deep shelf*", and limestone is settled in "*turbulent shallow water micro-facies environment*". The thickness of the formation varies between 100 – 250 m. There is contact with the overlying Fırat Formation.

H3.1.4 *Fırat Formation (TMF)*

The unit consists of limestone with reef characteristics in some places as a member of Midyat Formation (Fırat Member). The formation starts with limestone, cream-whitish off-yellow with medium-thick layers and without any layers in some places; and there is limestone that is off-yellow with medium-thick layers and with abundant chert knobs and abundant fossil shells on it. The upper layer consists of bioclastic limestone, cream-off-yellow in color, thick-very thick layers, low cherty knobs, and with abundant echinite, ostrea, gastropod and lamel. Limestone is settled in "*turbulent shallow water micro-facies environment*". The thickness of the formation varies between 0 – 150 m and there is a contact complying with Gaziantep formation.

H3.1.5 *Yavuzeli Basalt (Ty)*

Yoldemir (1987) has named this unit consisting of basalt lava whereas Tuna (1973) used the name Karacadag Basalt for the unit. Yavuzeli Basalt is reddish-dark grey and blackish in color, without any layers and with very thick layers in some places, calcite-filled porous, and generally consists of lava flow. These pyroclastics are especially observed on Gaziantep-Kilis road, near Kilis. No studies have been conducted on how the basalt is formed, and their exit areas. Researchers that previously worked in the region have various opinions on this issue. Some researchers relate the formation of this basalt to the East Anatolian Fault and relevant fault systems, and some others to the expansions due to the compression that started in Middle Eocene. The thickness of the basalt varies between 0 – 50 m. Yavuzeli Basalt dates back to Upper Miocene according to its stratigraphic location in the mapped area.

H3.1.6 *Old Alluvion (Qe)*

This layer generally consists of loose gravel, sand and mudstones in old river beds and plains surrounded by high hills. It is quaternary aged.

H3.1.7 *Geological Features of the Project Site*

Geological features of the Project Site were identified during the Soil Investigation Study (December 2014 – February 2016). The study showed that, the geological upper unit consists of top soil ranging in thickness between 0.5 and 3.0 m. In addition, fill material was present with a thickness of 10 m at two borehole locations representing a very limited area compared to the whole Project site; therefore, this was not taken into account while defining the general geological characteristics of the site.

The geological unit observed below the top and fill material (and also noted to be extruding on the site surface) was basalt containing decomposed tuffite. This is identified as the Yavuzeli Basalt (Ty) upper Eocene-aged young unit (Decomposed Tuffite and Basalt). This unit was seen to be continuous between a depth of 21 m and 31 m during the site investigations. The unit also continues in the surrounding area, as observed during the excavations. As basalt is the dominant unit in the subsoil, liquefaction is not seen as a potential problem. Moreover, as the unit is basalt there is no potential of subsiding, swelling and collapsing. Structural elements such as fissures, fractures and cracks, which were observed in some places in the rock unit, were noted to be discontinuous and irregular. Also, since the ground is mainly a competent rock unit, consolidation settlement is not expected. These rock features did not have the structure and properties to adversely impact the building foundations to be constructed. Limestone (i.e. beige in colour, medium-thick layered, with abundant chirt knobs) belonging to Firat Formation was observed at three borehole locations at the south and southeast of the Project site. The locations of the boreholes drilled during the Soil Investigation Study are illustrated in *Figure H3.3*, whereas *Figure H3.4* shows the selected cross-sections representing the geological features of the Project site.

Figure H3.3 Locations of the Boreholes

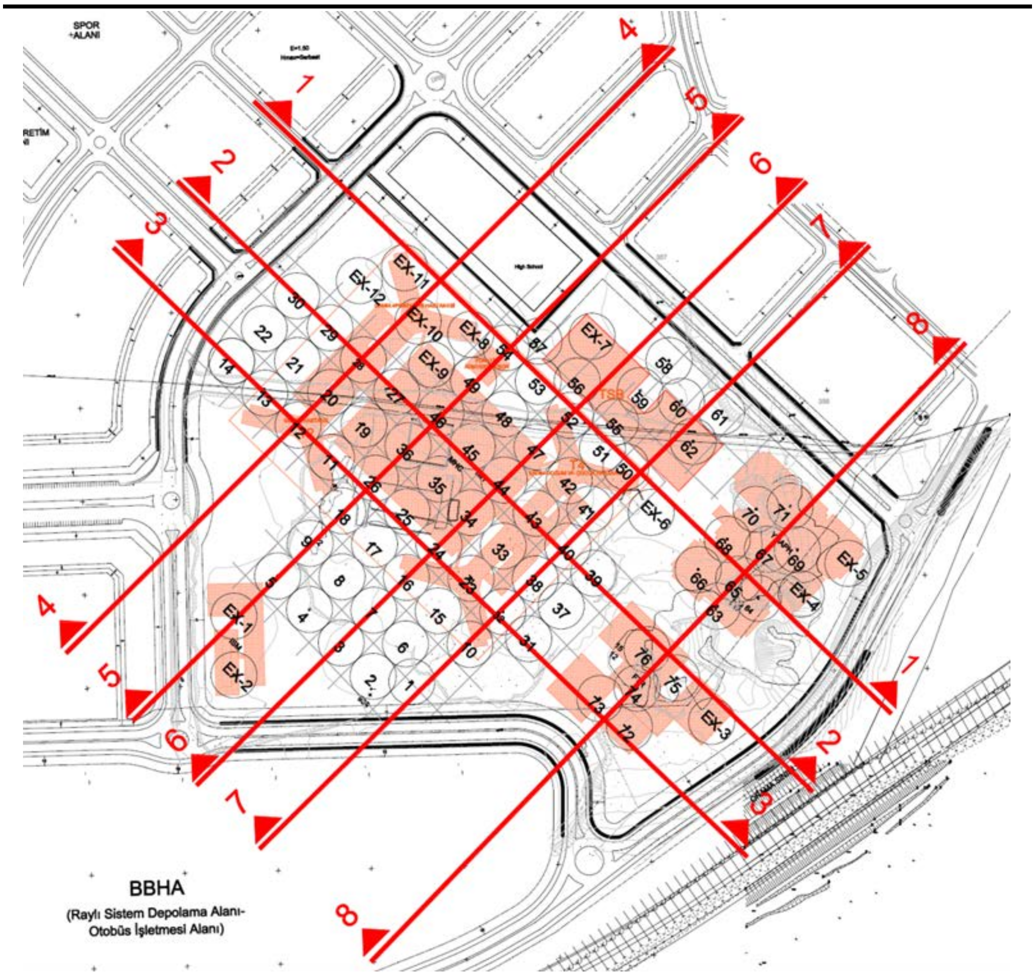
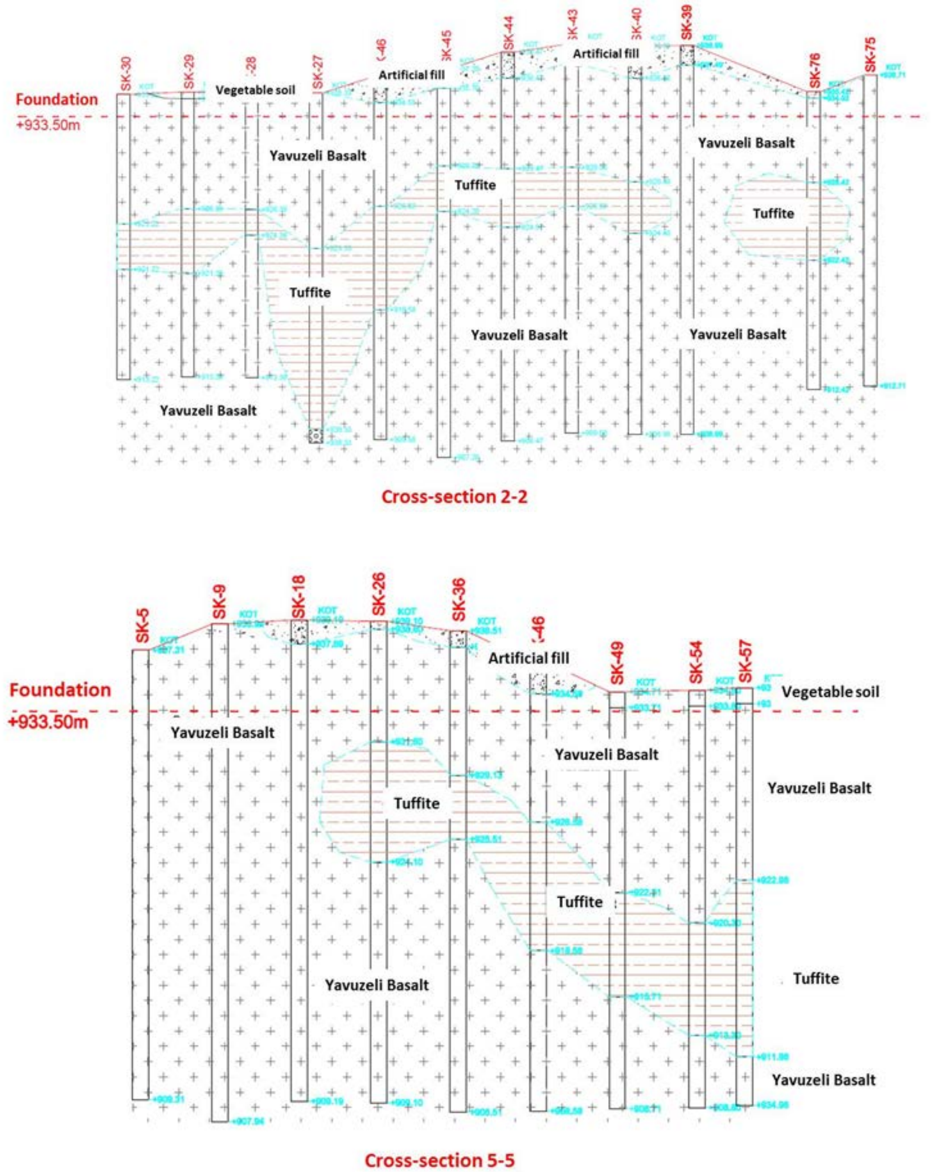


Figure H3.4 Selected Cross-Sections Representing the Site Geology



Geological layers were also distinguished in the Geophysical Surveys conducted at the Project Site. The following results were obtained for the site units:

- the Project Site is defined as 'Very steady' in terms of resistance,
- the Project Site is defined as 'Medium' compression based on the information on the resistance and durability of the rock unit,
- the basalt unit at the Project Site is defined to be 'Loose' at certain levels and 'Steady Rock' at certain levels, and
- ground density of Yavuzeli Basalt (Ty) upper Eocene-aged young unit (Basalt) is defined as 'Medium-High'.

Based on the above findings, the ground classification of the Project Site, according to TS EN 1998 – 1 (Eurocode 8) [1] and for all MASW points is defined as Class B – Very tight sand, gravel or very hard clay.

H3.1.8 *Seismic and Liquefaction Risks in the Region*

As mentioned in the Seismic Hazard (risk) Analysis Study, Turkey is a region where the Arabian platform and Asian platform collide, which resulted in development of an asymmetric tectonic drift system. It is characterised in a structure family where the major and largest ones in this tectonic system are represented with strike slip faults. The time elapsed from the most recent tectonic restructuring in the region is defined as Neo-tectonic Period. For this purpose, the collision of Anatolian and Arabian plate in Middle Miocene era is accepted as the beginning of this period.

The Major fault zones causing the dynamic behaviour in Turkey are the North Anatolia Fault Zone (NAFZ) and the East Anatolia Fault Zone (EAFZ). Gaziantep Province is located within the zone of influence of the seismically active EAFZ. The EAFZ starts from the Karlıova triple junction and continues to Turkoglu junction in the Southwest [2]. The fault continues towards the Mediterranean Sea. The fault that influences the structural evolution of the Gaziantep Basin is the Dead Sea Fault Zone (DSFZ) which is an active left lateral fault zone, approximately 1,000 km in length. DSFZ is bound by the Arabian and African Plates and continues in a northerly direction passing from Israel, Jordan, Lebanon, Syria and Turkey, whereas the southern direction reaches to the Red Sea slipping. DSFZ steps to the west of Gaziantep before merging with the Anatolian plate and African plate to form EAFZ. The Gaziantep Basin is located south of the 'Suture Zone' which was formed during the collisions of Arabian and Anatolian plates in late Cretaceous and Miocene times [3]. These two tectonic phases are indicated by the widespread occurrence of ophiolitic rocks on top of Cretaceous and Miocene formations.

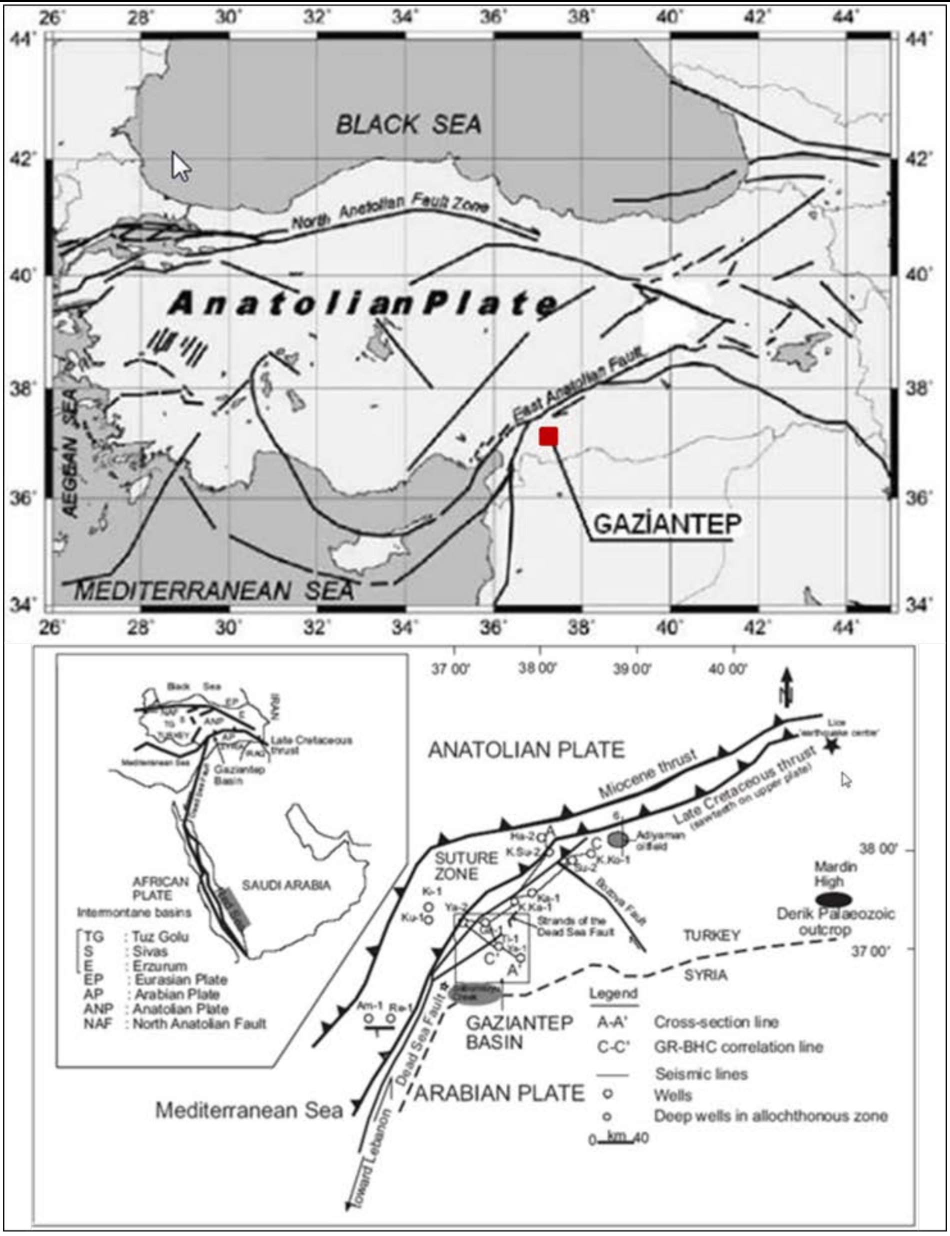
A simplified tectonic map of Turkey provided in *Figure H3.5* illustrates the major tectonic structures and plates influencing the structural evolution of Southeast Turkey as well as in the study area.

(1) Eurocode 8 - Design of structures for earthquake resistance.

(2) Gullu H., Ansal A.M. and Ozbay A. (2008). Seismic hazard studies for Gaziantep city in South Anatolia of Turkey. Nat Hazards, 44: 19 – 50

(3) Cabalar A.F. (2008). An Assessment of Earthquake Hazard in Gaziantep Turkey. EJGE Volume 13E

Figure H3.5 Tectonic Map of Turkey

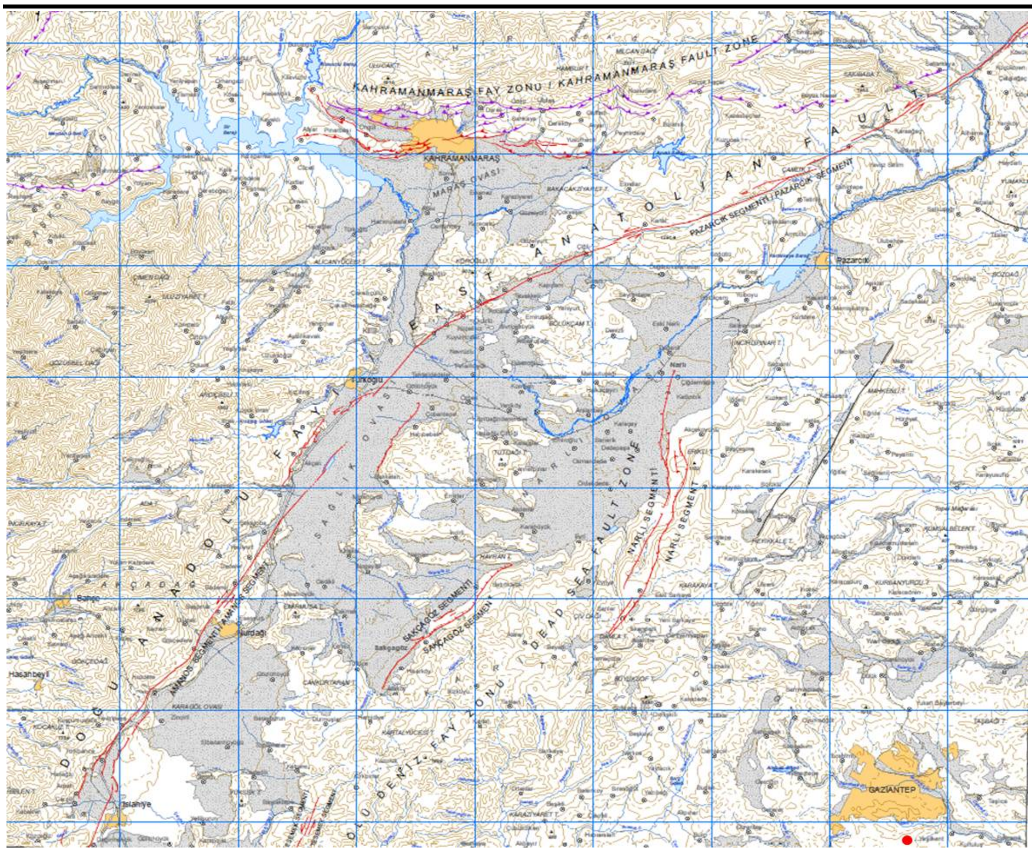


Source: Map modified from Cabalar, 2008 (1) - Gullu et al., 2008 (2)

The closest earthquake risk regions around the Gaziantep Central District are the Eastern Anatolia faults area located along Oludeniz, Reyhanli, Kirikhan, İslahiye, Turkoglu, Kahramanmaraş, Golbası and Adiyaman. The regional tectonic setting of Gaziantep is presented in *Figure H3.6*.

(1) Cabalar A.F. (2008). An Assessment of Earthquake Hazard in Gaziantep Turkey. EJGE Volume 13E
(2) Gullu H., Ansal A.M. and Ozbay A. (2008). Seismic hazard studies for Gaziantep city in South Anatolia of Turkey. Nat Hazards, 44: 19 – 50

Figure H3.6 Active Fault Map in the Vicinity of Gaziantep (red lines represents fault lines, red dot at the bottom right represents the approximate location of the Project site)

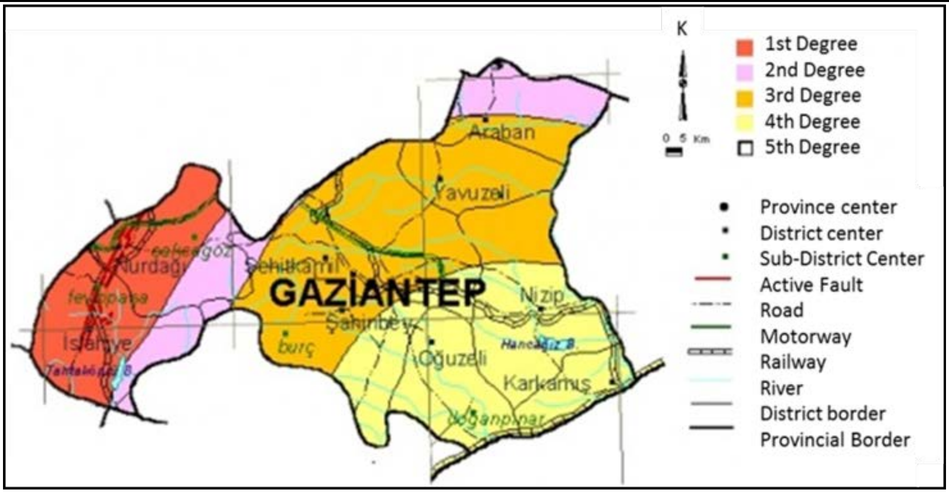


Source: http://www.mta.gov.tr/

As can be seen from *Figure H3.6,* the main fault EAFZ which has potential to create seismic hazard in Gaziantep Province is situated approximately 40-50 km to the northwest of central Gaziantep. In addition, there are some other smaller faults which may potentially create earthquakes and cause seismic hazard in Gaziantep. These include Bozova, Tut and Elbistan. The Bozova and Tut fault lines are right lateral strike slip faults and located to the north east of Gaziantep. The Elbistan fault is a right lateral strike slip fault 70 km in length and located north of Gaziantep. Gaziantep Central District is not directly located on any active fault.

The seismic zone classification map for Gaziantep Province is shown in *Figure H3.7.* It shows that Gaziantep Central District and the Project Site are located in a 3rd degree seismic zone. Hence, it is necessary to comply with the provisions of the regulations, principally the Regulation on Buildings to be built in Seismic Zones.

Figure H3.7 Seismic Zone Classification of Gaziantep Province



Source: Map of Turkey Seismic Zones, Ministry of Public Works and Settlement, General Directorate of Disaster Affairs, Earthquake Research Department, 1996

Since 2003, seismic activities on the EAFZ have increased, along with associated damage. Due to the seismic gap located on the nearest seismic EAFZ, (named Türkoğlu), the city of Gaziantep has not experienced intensive earthquakes for more than a century [(1)]. The distance of the Project Site to the Türkoğlu seismic gap is 60 km. As can be seen *Table H3.1*, Gaziantep Province has not experienced severe earthquakes. Historical earthquakes and recent major earthquakes (magnitude > 3.0 Mw) recorded in Gaziantep are listed in *Table H3.2*, respectively.

Table H3.1 Houses Damaged in Historical Earthquakes in Gaziantep Province

Year	Location	Number of Houses damaged
1971	Şahinbey, Sarısalkım	6
1971	Nurdağı, Gedikli	4
1971	Nurdağı, Koçkal	5
1971	Nurdağı, İçerisu	2
1971	Nurdağı, Satıröyük	1
1971	Nurdağı, Akınyolu	1
1971	Nurdağı, Hisar	1
1971	Nurdağı, Kurudere	2
1971	Nurdağı, Kozdere	104
1971	Nurdağı, Toplamalar	9
1971	Nurdağı, Gökçedere	60
1971	Nurdağı, Gözlüböyük	17
1971	İslahiye, Hasanlök	45
1980	İslahiye, Yelli Burun	31
1980	Nurdağı, Gökçedere	60
1986	Şehitkamil, Üçgöz	29
1986	Şehitkamil, Yeşilce	1
1986	Şehitkamil, Karadede	1

Source: General Directorate of Disaster Affairs official web site. (http://www.gaziantepafad.gov.tr/gaziantep-afetselligi)

(1) Gullu H., Ansal A.M. and Ozbay A. (2008). Seismic hazard studies for Gaziantep city in South Anatolia of Turkey. Nat Hazards, 44: 19 – 50

Table H3.2 Historical Earthquakes in Gaziantep Province Measuring Magnitude >3.0

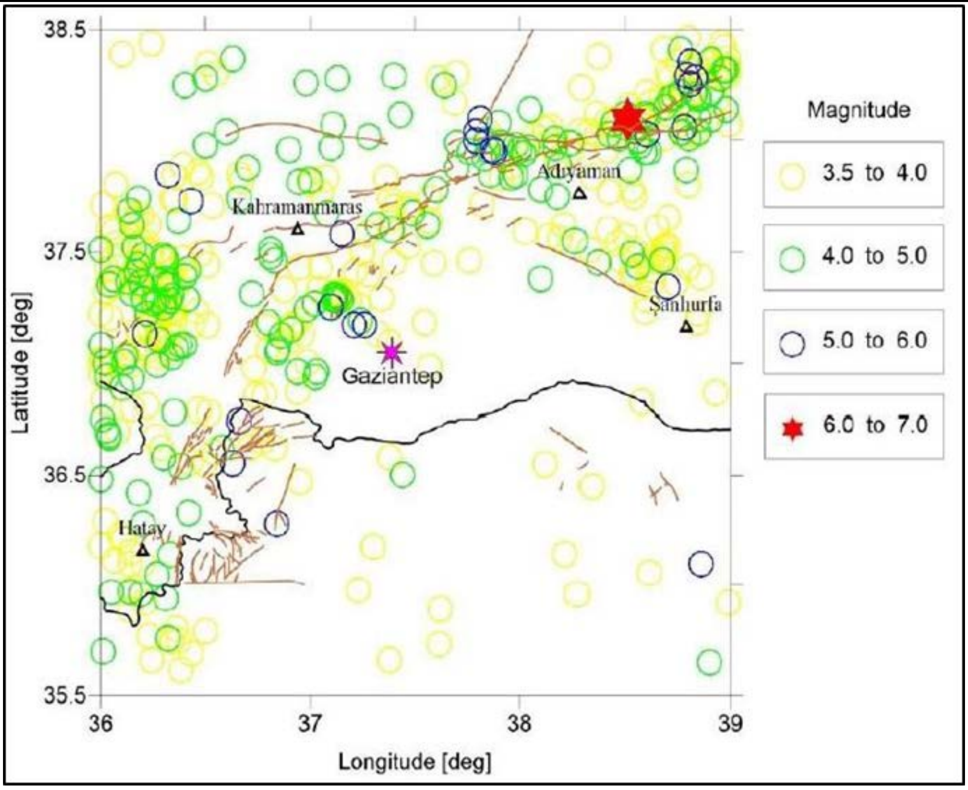
Date	Location	Magnitude (M_w)
08.01.2015	Nurdağı, Durmuşlar	4.6
14.07.2012	İslahiye, Şahmaran	3.9
28.08.2014	Nurdağı, Kuzoluk	3.9
26.02.2009	Nurdağı, Sakçagöze	3.8
10.04.2012	Nurdağı, Sakçagöze	3.6
05.05.2013	Şehitkamil, Eskisarkaya	3.5
17.02.2009	Nurdağı, Sakçagöze	3.4
31.04.2015	İslahiye, Yeniceli	3.4
31.10.2009	Nurdağı, Sakçagöze	3.3
01.02.2012	Nurdağı, Naimler	3.3
29.07.2012	Nurdağı, Nogaylar	3.3
01.04.2015	Nurdağı, Gozluhuyuk	3.3
15.04.2008	İslahiye	3.2
16.06.2008	Karkamış	3.2
06.12.2008	Nurdağı, Sakçagöze	3.2
01.04.2008	Şahinbey, Burç	3.1
26.08.2009	Nurdağı, Sakçagöze	3.1
01.11.2009	Nurdağı, Sakçagöze	3.1
21.01.2012	Nurdağı, Sakçagöze	3.1
06.04.2008	Nurdağı, Sakçagöze	3.0
22.08.2008	Şehitkamil	3.0
17.01.2009	İslahiye	3.0
07.07.2009	Şahinbey, Burç	3.0
17.01.2010	Nurdağı	3.0
24.12.2010	Oğuzeli	3.0
27.01.2011	Karkamış	3.0
08.09.2011	Şehitkamil	3.0
16.01.2012	Nurdağı, Naimler	3.0
17.07.2012	İslahiye, Akınyolu	3.0

Source: http://www.depremler.org/en-buyuk-depremler-gaziantep

The most recent major earthquake in the region occurred on 8th January, 2015 in Nurdağı, Durmuşlar Village with a magnitude of 4.6 Mw, 8 km below surface level. According to press announcements made by the Turkish Republic Prime Ministry Disaster and Emergency Management Presidency, no injuries were recorded and no severe damage to buildings occurred [(1)]. The distribution of earthquakes in Gaziantep region based on data from KOERI (Bogazici University, Kandilli Observatory and Earthquake Research Institute) for between 1910 and 2015, in terms of magnitude and depth are presented in *Figure H3.8* and *Figure H3.9*.

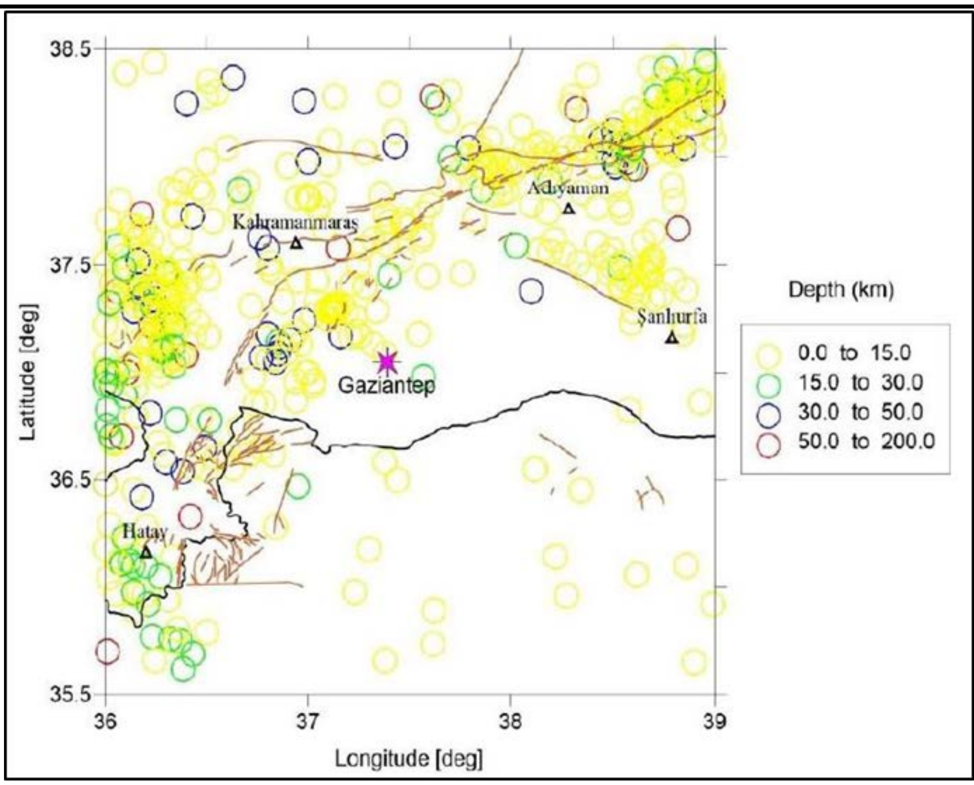
(1) https://www.afad.gov.tr/tr/HaberDetay.aspx?IcerikID=3334&ID=12

Figure H3.8 Distribution of Earthquakes in the Region According to the Magnitude (1910 – 2015)



Source: Seismic Hazard (Risk) Analysis Report for Gaziantep IHC Project (June 2015) based on data from KOERI

Figure H3.9 Distribution of Earthquakes in the Region According to the Depth



Source: Seismic Hazard (Risk) Analysis Report for Gaziantep IHC Project (June 2015) based on data from KOERI

According to *Figure H3.8* and *Figure H3.9*, approximately 78% of the earthquakes in the region occurred at depths of between 0-15 km and 93 % of the earthquakes have had a magnitude below 5 Mw.

H3.1.9 *Seismicity and Earthquake Hazard (Risk) Analysis of the Project Site*

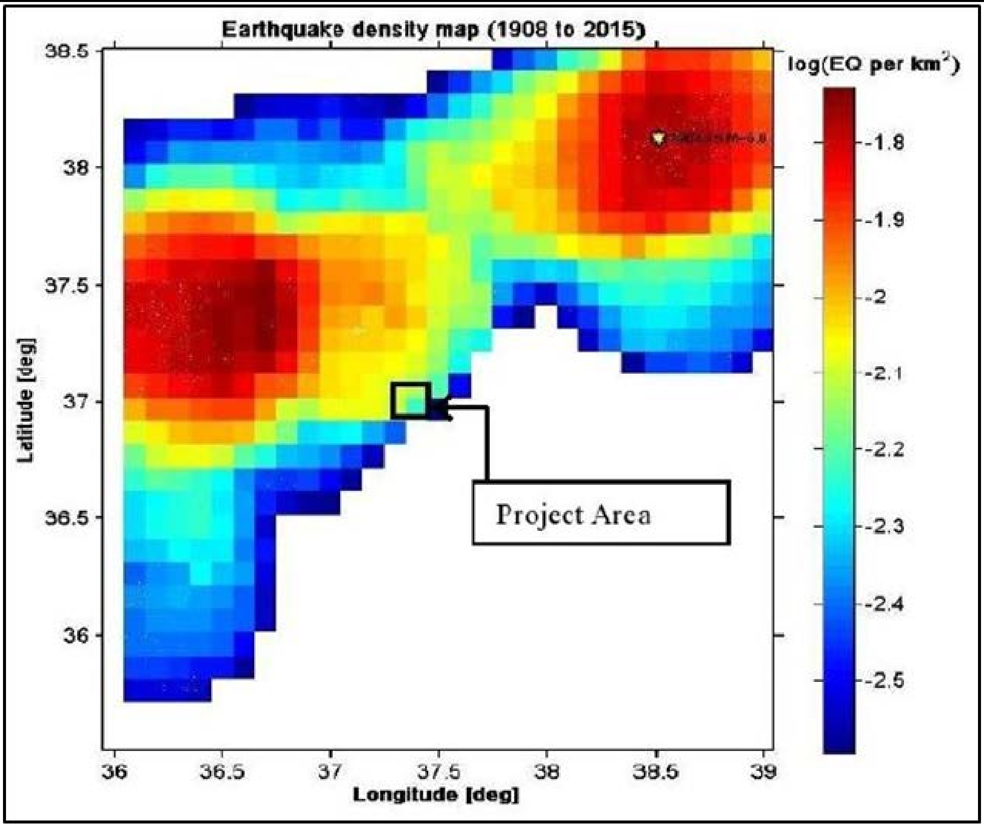
A Seismic Hazard (Risk) Analysis Study was conducted in June 2015 for the buildings footprint within the Project Site. The study determined the probability of seismic hazards together with the preparation of the spectra recommended for use during the design for the Project.

The seismic risk within a 100 km radius of the Project Site was analysed using data for earthquakes with magnitude of 4.5 Mw and higher, that occurred between 1910 and 2015.

In addition, a second analysis was made to statistically determine the earthquake with the highest magnitude by using earthquakes with magnitude of more than 3.5 Mw included in the earthquake catalogue by KOERI between 1910-2015.

As a result of the first analysis, earthquake intensity and seismicity maps were created for the Project Site and possible effects were analysed. Results showed that high (red) and low (blue) 'b' values (i.e. a seismotectonic parameter which changes depending on the tectonic characteristics of an area) of earthquake intensity were at least 50-60 km away from the Project Site, which shows that the significance of earthquake risk is lower at the Project Site. The earthquake intensity distribution in the region (including the Project Site) is presented in *Figure H3.10*.

Figure H3.10 Earthquake Intensity Distribution in the Region



Source: Seismic Hazard (Risk) Analysis Report for Gaziantep IHC Project (June 2015)

Moreover, the second analysis revealed that the highest earthquake magnitude that can be observed in the region is in the range of 6.15 – 6.2 Mw. However, it is suggested that a magnitude of 6.5 Mw be used in order to increase safety.

In addition, soil profile classification was undertaken during in the Seismic Hazard (Risk) Analysis study. Accordingly, although core samples taken from the Project Site seem to be high in terms of faults, the Vs30=720 m/sec (Vs30 refers to the average shear-velocity down to 30 m) value obtained from Multispectral Analysis of Surface Waves (MASW) measurements show that the ground has rock soil characteristics suitable for engineering structures. *Table H3.3* below presents the soil type classification for seismic amplifications.

Table H3.3 Soil Profile Type Classification for Seismic Amplification

Soil Type (National Earthquake Hazards Reduction Program, NEHRP)	General Description	Average Shear Wave velocity to 30 m (m/s)
A	Hard rock	>1500
B	Rock	760<Vs≤1500
C	Very dense soil and soft rock	360<Vs≤760
D	Stiff soil 15≤N≤50 or 50 kPa ≤Su≤100 kPa	180≤Vs≤360
E	Soil or any profile with more than 3 m of soft clay defiled as soil with PI>20, w≥40%, and su<25kPa	≤180
F	Soils requiring site specific evaluations	-

Source: Building Seismic Safety Council. 2003, NEHRP Recommended Provisions for seismic Regulations for New buildings and other Structures

The Project Site corresponds to C-B class and is defined as "*very dense soil and soft rock*".

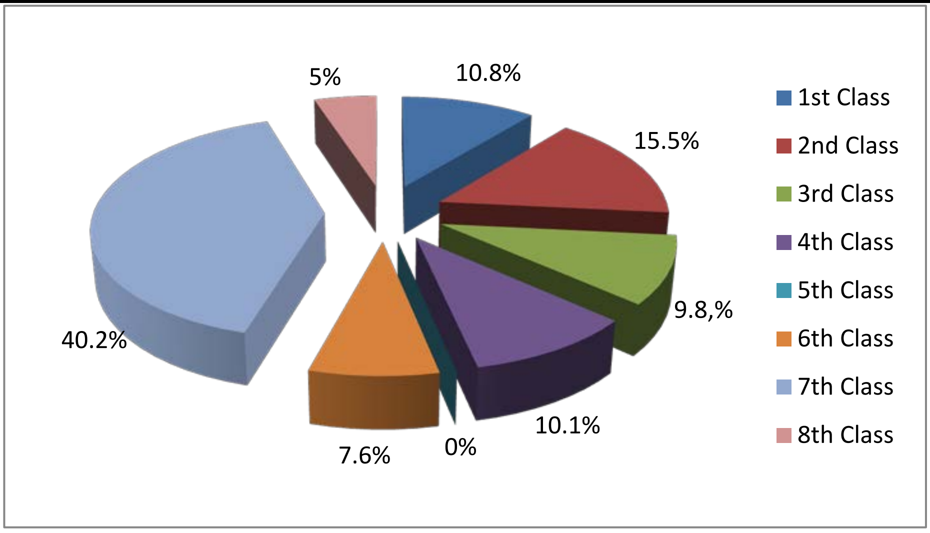
The results of the Seismic Hazard (Risk) Analysis for the Project Site can be summarised as follows:

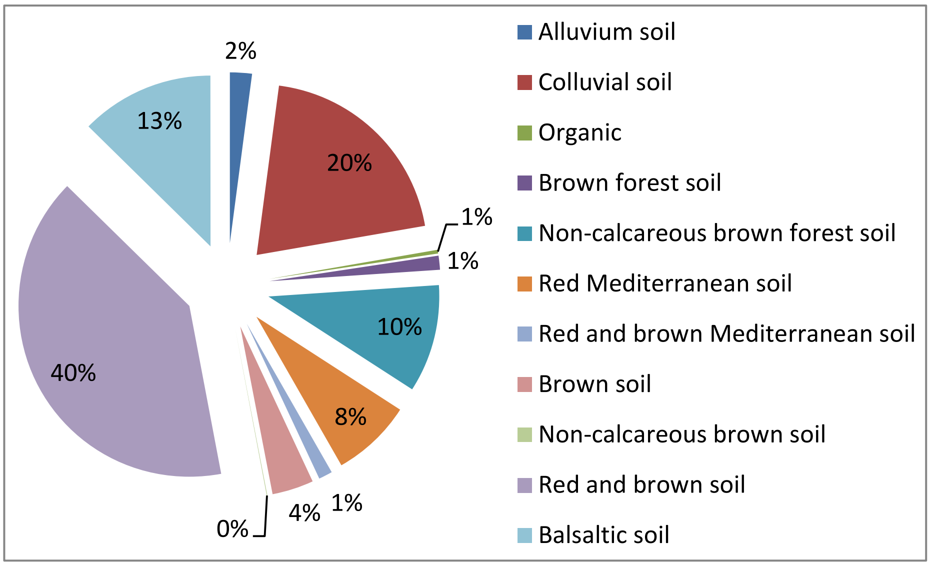
- the Project Site is approximately 50 km east of the active EAFZ;
- the greatest earthquake intensity in the region is found to be at least 50-60 km away from the Project Site and this means the significance of earthquake risk is lower at the Project Site;
- the highest earthquake magnitude that could be observed in the region is predicted to be 6.5 Mw;
- the soil profile of the Project site is classified as C-B Class which is defined as "*very dense soil and soft rock*";
- spectral acceleration, velocity and displacement values for the study area are defined in the Soil Investigation Report and they will be complied with during the Project design studies; and
- natural disasters such as landslides, rock falls, avalanche and flooding are not expected at the Project Site.

H3.2 SOIL STRUCTURE OF GAZIANTEP PROVINCE

Soils are classified into eight classes depending on their quality; the first class refers to the soils which do not have the risk of erosion and are suitable for easy and economic agricultural activity; whereas the eighth class refers to soils which are not suitable for agriculture and can only be used as recreational areas. The soil classes and structure for land in Gaziantep Province are shown in *Figure H3.11*.

Figure H3.11 Distribution of Land Classes and Soil Structure in Gaziantep Province





Source: Gaziantep Environmental Status Report, 2014

As stated in the Gaziantep Environmental Status Report (2014) due to the fact that climate and physical conditions of the region differ, soil structure varies greatly in Gaziantep.

The surface of the Project Site is mainly fill material or extrusion of rock units from the Yavuzeli Basalt unit.

H3.2.1 Site Soils and Contaminated Land

It was observed during the scoping visit that the local municipality has been temporarily storing domestic waste in the southwest corner of the site. It is reported by the SPV that the municipality has stopped storing domestic waste on the Project Site and that the site has now been fenced off with necessary signs in place to indicate that the site is designated for the Project. It was further reported by the SPV that these waste materials have been removed. No soil sampling study was undertaken as part of the ESIA due to the ground of the Project Site having a rock structure and there being a very low potential for contamination to leak from the domestic waste found on site (*see Appendix H1*). The removal of this waste is therefore the appropriate measure to be taken.

H4 IMPACTS

H4.1 POTENTIAL IMPACTS DURING CONSTRUCTION

H4.1.1 *Impacts Relating to Geology and Seismic Risks*

In the event of an earthquake during construction, significant impacts on the environment as well as on the community and workers' health and safety may arise following accidents, spills, fire, etc. related to the seismic incident. The Project Site lies within a 3rd degree seismic zone and a site specific seismic hazard analysis study has been conducted. Accordingly, the Project design will take into account the relevant Turkish regulatory requirements relating to seismic design and risk assessment and also the findings of the site specific seismic hazard analysis study. During all construction works within the Project Site, the Regulation on Buildings to be Built in Seismic Zones (Official Gazette date/no: 06.03.2007/26454) will be complied with. Based on this, the risks are considered to be as low as technically and financially feasible. Therefore, the magnitude of impact can be considered between negligible to small. Since the Project site lies within the 3rd degree seismic zone, the Project site sensitivity is medium and the impact related to geology and seismic risks is found to vary between **negligible** and **small**. In addition, it is important to note that slope stability will be ensured during excavation works on the Project Site and necessary safety measures will be taken to minimise impacts relating to soil stability and excavation.

H4.1.2 *Impacts on Soils*

Temporary use of land for construction, if not properly managed can lead to impacts on soil quality as a result of events such as compaction and accidental spills of liquid cement (excluding hazardous material spills). Construction activities and storage of construction equipment and materials on soils also have the potential to affect soil through spills of hazardous material such as oils, fuel or other materials (i.e. during fuel loading for machinery operating at the site). These aspects will be managed through the following mitigation measures that are embedded in the Project design:

- All contractors will be required to adopt good construction site practices for the protection of soils and to follow the General IFC EHS Guidelines.
- Provisions will be taken for the protection of newly exposed soil surfaces from rainfall and wind erosion such as silt fences.
- The use of cement and wet concrete in or close to any exposed areas will be carefully controlled.
- Fuels, oils and chemicals will be stored on an impervious base protected by bunds of 110% of capacity of the largest tank/container. Drip trays will

be used for fuelling mobile equipment. Any spillages from handling fuel and liquids will be immediately contained on site. The contaminated soil will be removed from the site for suitable treatment and disposal in an appropriately licensed disposal site.

- Spoil and other surplus material arising from construction works which is classed as 'acceptable fill' will, wherever practicable, be recovered and used in the construction works. Relevant authorities will be consulted regarding this on a case by case basis to ensure the re-use of waste materials is acceptable. In addition, surplus construction material will be made available to third parties for reuse on local development projects if it cannot be utilised on site.

The vulnerability of soil is considered low since there is no agricultural activity in the Project Site. Provided that the good construction practices and above-mentioned embedded measures are applied to provide protection against soil the magnitude of the impacts is considered small. Therefore, the resulting impacts are expected to be **negligible**.

H4.2 POTENTIAL IMPACTS DURING OPERATION

H4.2.1 Impacts Relating to Geology and Seismic Risks

The SPV has included earthquake-resistant design details within the Structural report of the Schematic design. These designs are still subject to approval by the MoH and their design and engineering consultants. The specifications for earthquake-resistant design in Turkey are provided in *Appendix H2* and the Project structural calculations and design are detailed in *Appendix H3*. The SPV conducted an earthquake assessment and found the building design to cover required seismic loads (see *Appendix H4*). The assessment of seismic load and earthquake-resistant design is outside the scope of this ESIA and therefore this ESIA report does not offer any recommendations or conclusions on the seismic load assessment detailed in *Appendix H4*, which is provided for reference only. In the event of an earthquake during operation, impacts to soils may arise following accidents, spills, fire, etc. related to the seismic incident. The necessary design steps will be followed during construction and compliance with regulatory requirements met as described in *Section 8.4.1*, the impact related to geology and seismic risks is found to vary between **negligible** and **small** (based on explanations given in *Section 8.4.1*).

H4.2.2 Impacts on Soils

During operation, soils could become contaminated from accidental spills of hazardous materials, accidental leakage from underground pipes used for sanitary wastewater discharges. There are specific mitigation measures embedded in the design of the Project. Fuels, oils and chemicals will be stored on an impervious base protected by bunds of 110% of capacity of the largest container. Drip trays will be used for fuelling mobile equipment. Any spillages from handling fuel and liquids will be immediately contained on site and the contaminated soil removed from the site for suitable treatment and disposal in an appropriately licensed disposal site.

The Project lies on land with no agricultural activity; therefore, the sensitivity is defined as low. The magnitude of the impacts is considered small considering the above mentioned practices that will be applied during operation. Consequently, the impacts are classified as **negligible**.

H4.3 MITIGATION MEASURES

In addition to the embedded mitigations described above, the following mitigation measures for protection of soil media during construction and operation phases will include the following:

- Hazardous and non-hazardous materials and waste during construction will be handled according to the Environmental and Social Management System to be prepared by SPV and where needed, further site-specific management plans will be developed (i.e. Hazardous Material Management Plan). Details of waste generation and management methods are provided in *Volume II, Annex E, Waste*.

- Operation of a drainage system and implementation of Emergency Preparedness and Response Plan in the event of spills, fire etc. will prevent significant impacts on soils.

H4.4 RESIDUAL IMPACTS

With the implementation of mitigation measures mentioned above, no significant residual impacts are expected during construction or operation.

H4.5 CUMULATIVE IMPACTS

No cumulative impacts relating to geology and seismic risks have been identified.

Appendix H1

Reasons for Not Undertaking Soil Sampling

H1-1 REASONS FOR NOT UNDERTAKING A SOIL SAMPLING STUDY

There are a number of reasons why it was not considered necessary to conduct a soil sampling survey to determine the presence of ground contamination. The manner in which the waste was previously stored on the site is unlikely to have resulted in the generation of leachate. This theory is reinforced by the lack of evidence of leachate pools present in the areas used for waste storage. At the Project site, it was observed during the Soil Investigation Study that the geological upper unit consisted of top soil; and in addition, fill material was only observed in a very limited area. The geological unit observed below the top and fill material was basalt. The ground can be described as being mainly a competent rock unit.

Leachate is usually generated during anaerobic degradation of waste, however the waste stored at the Project site is thought to have been in the aerobic degradation phase throughout the time that it was present on the site. Usually in full-scale landfills, the aerobic degradation phase is generally of limited duration due to the high oxygen demand of waste relative to the limited quantity of oxygen present inside a landfill (Landfilling of Waste: Leachate, 1997) [(1)]. However, in the case of the Gaziantep IHC Project, due to the small height of the waste material and it not being in a confined area and the abundance of oxygen, aerobic degradation is likely to have persisted for the duration that the waste was present on the site. Usually, in a full-scale landfill the only layer involved in aerobic metabolism is the upper layer where oxygen is trapped in fresh waste and is supplied by diffusion and rainwater, however in the case of the waste stored on the Project site this is thought to have been the case for the whole of the stockpile. There was no evidence of leachate pools at the site where the waste material was placed as observed during a site visit in April 2015.

Additionally, considering the geology of the area on which the waste was stored and the small potential for leachate generation, the risk of rock contamination decreases. Other factors reducing the risk of contamination are outline below:

- There is no fill material where the waste is placed as seen in the cross sections and boring logs (see *Figure H1-1.3* and *Figure H1-1.4*). The material is basaltic rock which represents a fractured media rather than soil material and which has uniform porosity and large areas of soil surface that would allow potential contaminants to be absorbed on to the surface. The rock cores shown in *Figure H1-1.4* below illustrate a good rock competency, indicating that the rock is not strongly weathered and the fractures through which any potential contaminated soil could migrate downward is limited. Limited fractures mean limited areas on which potential contaminants can attach.

(1) Landfilling of Waste: Leachate, Christensen, T.H., Cossu, R., Stegmann, R., E&FN SPON and imprint of Chapman&Hall, UK, 1997.

- From an extensive literature review, there was little information on the potential for basaltic rocks to adsorb contaminated materials, specifically related to water infiltration into the subsurface from waste material. In addition, the fact that the natural rock material may have its own highly heterogeneous metal concentrations, it was deemed difficult to be able to quantify the actual potential for contamination from surface water infiltration.

In summary, it was deemed that the risk of rock contamination from the presence of waste on the Project site was considered to be small due to:

a) the small potential for leachate generation;

b) the small infiltration occurrence of the contaminated surface water into the subsurface due to the rock competency;

c) the small area of potential absorption on the rock surface for contaminants; and

d) potentially high variability of the original rock material metal concentrations.

It was also technically challenging to quantify whether metal contamination was present.

It should also be pointed out that there was no presence of water detected in any of the soil borings at the site. Lack of a water table to the depths measured at the site also significantly decreases potential to impact the water table.

The locations of the boreholes drilled within the scope of the geotechnical investigation, the location of the waste materials within the Project site, the cross sections of boreholes drilled around the waste material storage area and the pictures of the borehole log samples for borehole log no:64 are illustrated below.

Figure H1-1.1 Locations of Boreholes Drilled within the Scope of Geotechnical Investigation

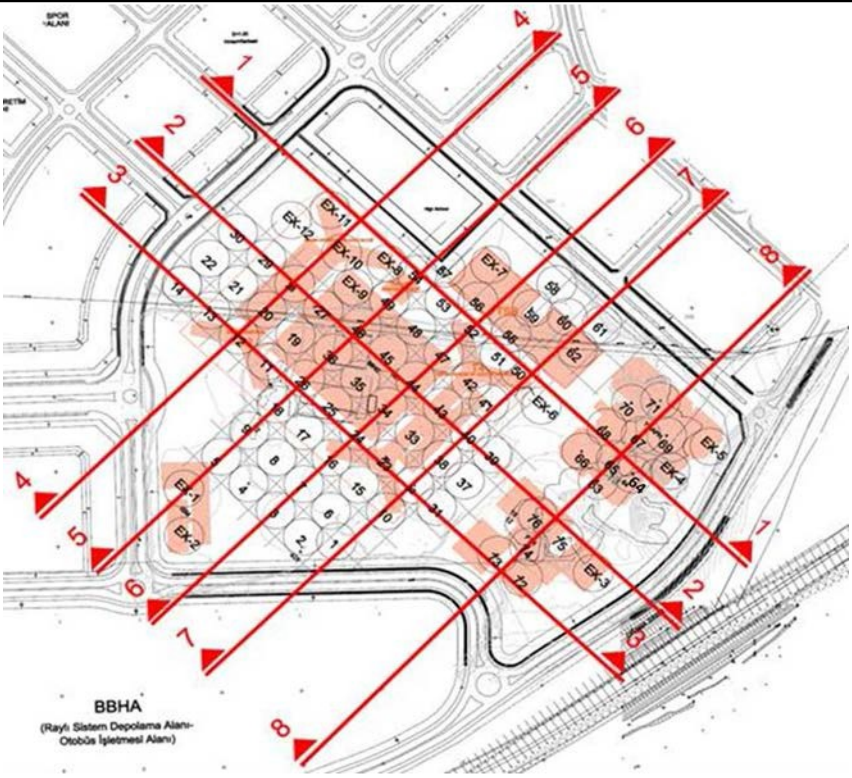
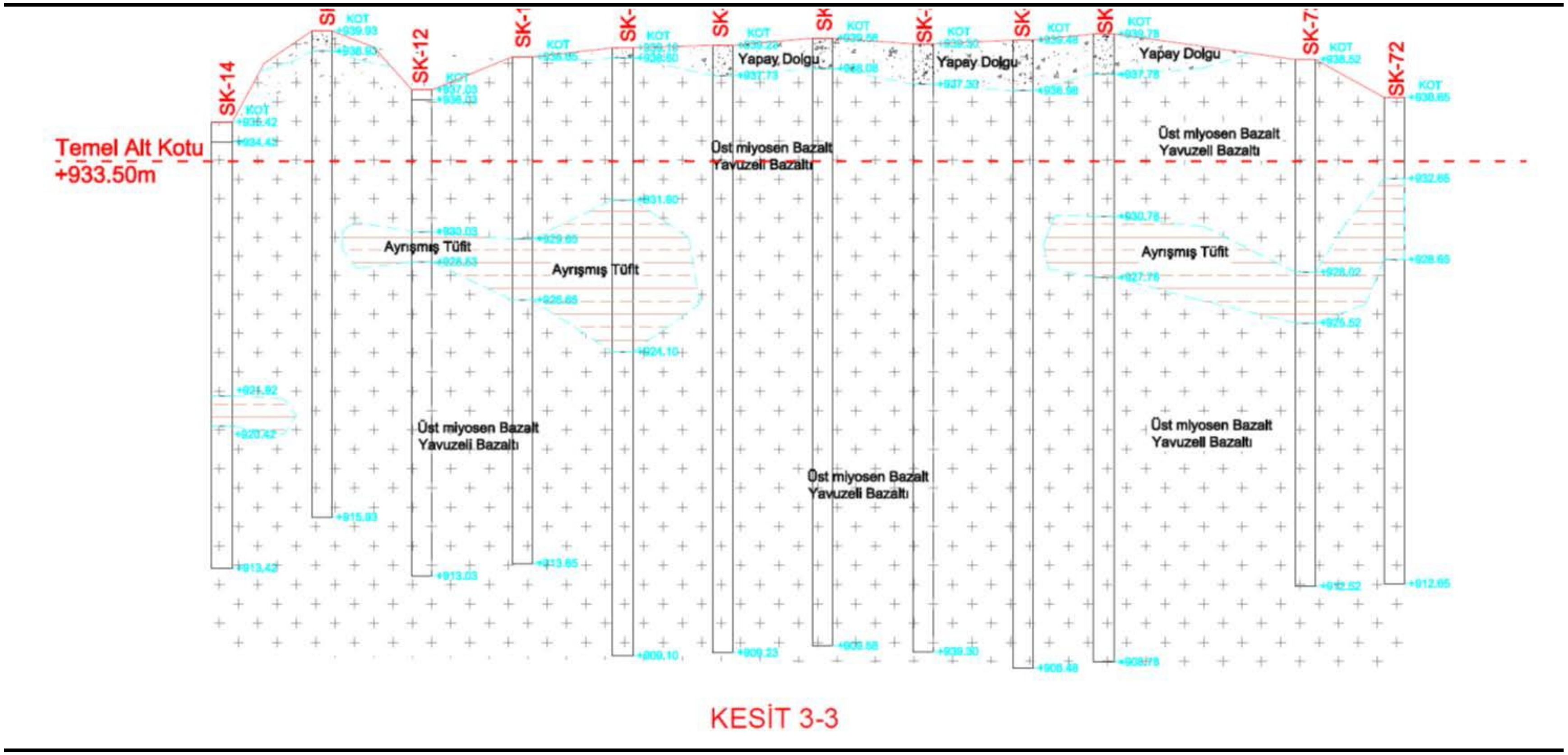
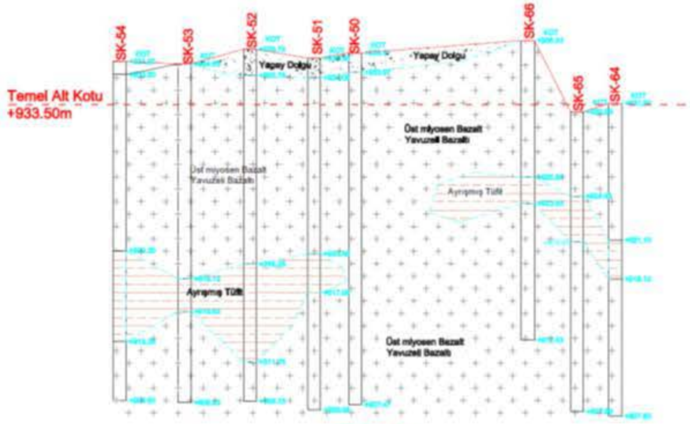


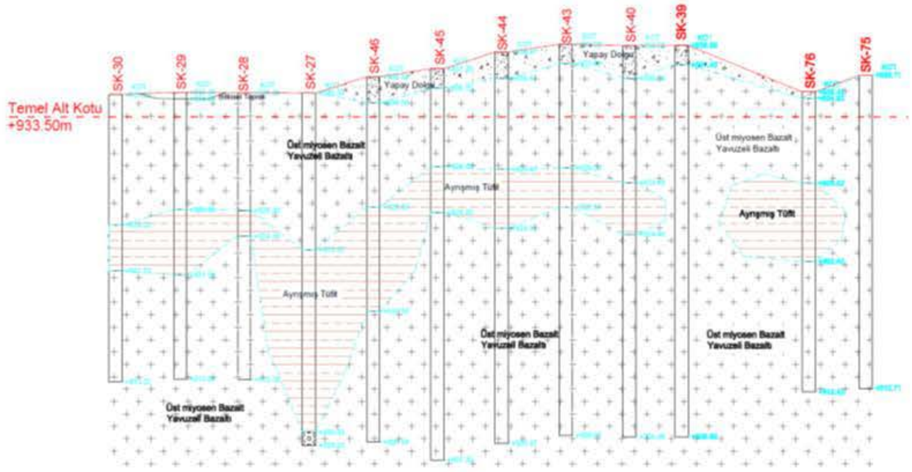
Figure H1-1.2 Location of the Waste Materials within the Project Site

*Figure H1-1.3 Cross Sections of Boreholes Drilled around the Waste Material Storage Area**





KESİT 1-1



KESİT 2-2

*Yapay dolgu: Artificial fill
Ayrışmış tüfit: Separated Tuffite
Üst miyosen Bazalt: Upper Miocene Basalt
Yavuzeli Bazalt: Yavuzeli Basalt

Figure H1-1.4 Pictures of the Borehole Log Samples for Borehole Log no: 64

Appendix H2

Specification for Buildings to be Built in Seismic Zones

Ministry of Public Works and Settlement
Government of Republic of Turkey

Specification for Buildings to be Built in Seismic Zones (2007) Chapters 1,2

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ENGLISH TRANSLATION
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REQUIREMENTS FOR BULDINGS TO BE BUILT IN SEISMIC ZONES

CHAPTER 1 – GENERAL REQUIREMENTS

1.1. SCOPE

1.1.1 – Requirements of this Specification shall be applicable to newly constructed buildings in seismic zones as well as to existing buildings previously constructed.

1.1.2 – Requirements applicable to existing buildings, which are subject to modification in occupancy and/or structural system and those to be assessed and retrofit before or after an earthquake are given in **Chapter 7**.

1.1.3 – Requirements of this Specification shall be applicable to reinforced concrete (cast-in-situ and prestressed or non-prestressed prefabricated) buildings, structural steel and masonry buildings and building-like structures.

1.1.4 – Until relevant code requirements are enforced, the minimum requirements and rules to be applied to timber buildings and building-like structures shall be determined by the Ministry of Public Works and Settlement and the designs shall be made accordingly.

1.1.5 – In addition to buildings and building-like structures, non-building structures permitted to be designed in accordance with the requirements of this Specification are limited with those specified in **2.12** of **Chapter 2**. In this context bridges, dams, harbour structures, tunnels, pipelines, power transmission lines, nuclear power plants, natural gas storage facilities, underground structures and other structures designed with analysis and safety rules that are different than those for buildings are outside the scope of this Specification.

1.1.6 – Requirements of this Specification shall not be applied to buildings equipped with special system and equipment between foundation and soil for the purpose of isolating the building structural system from the earthquake motion, and to buildings incorporating other active and passive control systems.

1.1.7 – Requirements to be applied to structures which are outside the scope shall be specifically determined by the Ministries supervising the constructions based on contemporary international standards and such structures shall be designed to those requirements until their own special specifications are prepared.

1.2. GENERAL PRINCIPLES

1.2.1 – The general principle of earthquake resistant design to this Specification is to prevent structural and non-structural elements of buildings from any damage in low-intensity earthquakes; to limit the damage in structural and non-structural elements to repairable levels in medium-intensity earthquakes, and to prevent the overall or partial collapse of buildings in high-intensity earthquakes in order to avoid the loss of life. The performance criteria to be considered in assessment and retrofit of existing buildings are defined in **Chapter 7**.

1.2.2 – The design earthquake considered in this Specification corresponds to *high-intensity* earthquake defined in **1.2.1** above. For buildings with Building Importance Factor of I = 1 in accordance with **Chapter 2**, **Table 2.3**, the probability of exceedance of the design earthquake within a period of 50 years is 10 %. Earthquakes with different probabilities of exceedance are defined in **Chapter 7** to be considered in assessment and retrofit of existing buildings.

1.2.3 – Seismic zones cited in this Specification are the first, second, third and fourth seismic zones depicted in *Seismic Zoning Map of Turkey* prepared by the Ministry of Public Works and Settlement and issued by the decree of the Council of Ministers.

1.2.4 – Buildings to be constructed to this Code shall follow the material and workmanship requirements of "General Technical Specification" of Ministry of Public Works and Settlement.

CHAPTER 2 – ANALYSIS REQUIREMENTS FOR EARTHQUAKE RESISTANT BUILDINGS

2.0. NOTATION

$A(T)$ = Spectral Acceleration Coefficient
A_o = Effective Ground Acceleration Coefficient
B_a = Design internal force component of a structural element in the direction of its principal axis a
B_{ax} = Internal force component of a structural element in the direction of its principal axis a due to earthquake in x direction
B_{ay} = Internal force component of a structural element in the direction of its principal axis a due to earthquake in y direction perpendicular to x direction
B_b = Design internal force quantity of a structural element in principal direction b
B_{bx} = Internal force component of a structural element in the direction of principal axis b a due to earthquake in x direction
B_{by} = Internal force component of a structural element in the direction of principal axis a due to earthquake in y direction perpendicular to x direction
B_B = Any response quantity obtained by modal combination in the Mode-Superposition Method
B_D = Amplified value of B_B
D_i = Amplification factor to be applied in Equivalent Seismic Load Method to ± %5 additional eccentricity at i'th storey of a torsionally irregular building
d_{fi} = Displacement calculated at i'th storey of building under fictitious loads F_{fi}
d_i = Displacement calculated at i'th storey of building under design seismic loads
F_{fi} = Fictitious load acting at i'th storey in the determination of fundamental natural vibration period
F_i = Design seismic load acting at i'th storey in Equivalent Seismic Load Method
f_e = Equivalent seismic load acting at the mass centre of the mechanical and electrical equipment
g = Acceleration of gravity (9.81 m/s^2)
g_i = Total dead load at i'th storey of building
H_i = Height of i'th storey of building measured from the top foundation level (In buildings with rigid peripheral basement walls, height of i'th storey of building measured from the top of ground floor level) [m]
H_N = Total height of building measured from the top foundation level (In buildings with rigid peripheral basement walls, total height of building measured from the top of the ground floor level) [m]
H_w = Total height of structural wall measured from the top foundation level or top of the ground floor level
h_i = Height of i'th storey of building [m]
I = Building Importance Factor
ℓ_w = Plan length of structural wall or a piece of coupled wall
M_n = Modal mass of the n'th natural vibration mode
M_{xn} = Effective participating mass of the n'th natural vibration mode of building in the x earthquake direction considered
M_{yn} = Effective participating mass of the n'th natural vibration mode of building in the y earthquake direction considered
m_i = i'th storey mass of building ($m_i = w_i / g$)
$m_{\theta i}$ = With floors are modelled as rigid diaphragms, mass moment of inertia around vertical axis passing through mass centre of i'th storey of a building
N = Total number of stories of building from the foundation level (In buildings with rigid peripheral basement walls, total number of stories from the ground floor level)

n = Live Load Participation Factor
q_i = Total live load at i'th storey of building
R = Structural Behaviour Factor
R_{alt},$R_{üst}$= R factors specified for stories below and the roof, respectively, in the case where single-story frames with hinged columns at the top are used as roofs of cast-in-situ reinforced concrete, precast or structural steel buildings
$R_{NÇ}$ = Structural Behaviour Factor defined in **Table 2.5** for the case where entire seismic loads are carried by frames of nominal ductility level
R_{YP} = Structural Behaviour Factor defined in **Table 2.5** for the case where entire seismic loads are carried by walls of high ductility level
R_a(T) = Seismic Load Reduction Factor
$S(T)$ = Spectrum Coefficient
$S_{ae}(T)$ = Elastic spectral acceleration [m /s^2]
$S_{aR}(T_n)$ = Reduced spectral acceleration for the n'th natural vibration mode [m /s^2]
T = Building natural vibration period [s]
T_1 = First natural vibration period of building [s]
T_A ,T_B = Spectrum Characteristic Periods [s]
T_m , T_n = m'th and n'th natural vibration periods of building [s]
V_i = Storey shear at i'th storey of building in the earthquake direction considered
V_t = In the Equivalent Seismic Load Method, total equivalent seismic load acting on the building (base shear) in the earthquake direction considered
V_{tB} = In the Mode-Superposition Method, total design seismic load acting on the building (base shear) obtained by modal combination in the earthquake direction considered
W = Total weight of building calculated by considering Live Load Participation Factor
w_e = Weight of mechanical or electrical equipment
w_i = Weight of i'th storey of building by considering Live Load Participation Factor
Y = Sufficient number of natural vibration modes taken into account in the Mode-Superposition Method
α = Coefficient used for determining the gap size of a seismic joint
α_S = Ratio of the sum of shears at the bases of structural walls of high ductility level to the base shear of the entire building
β = Coefficient used to determine lower limits of response quantities calculated by Mode-Superposition Method
Δ_i = Reduced storey drift of i'th storey of building
$(\Delta_i)_{ort}$ = Average reduced storey drift of i'th storey of building
ΔF_N = Additional equivalent seismic load acting on the N'th storey (top) of building
δ_i = Effective storey drift of i'th storey of building
$(\delta_i)_{max}$ = Maximum effective storey drift of i'th storey of building
η_{bi} = Torsional Irregularity Factor defined at i'th storey of building
η_{ci} = Strength Irregularity Factor defined at i'th storey of building
η_{ki} = Stiffness Irregularity Factor defined at i'th storey of building
Φ_{xin} = In buildings with floors modelled as rigid diaphragms, horizontal component of n'th mode shape in the x direction at i'th storey of building
Φ_{yin} = In buildings with floors modelled as rigid diaphragms, horizontal component of n'th mode shape in the y direction at i'th storey of building
$\Phi_{\theta in}$ = In buildings with floors modelled as rigid diaphragms, rotational component of n'th mode shape around the vertical axis at i'th storey of building
θ_i = Second Order Effect Indicator defined at i'th storey of building

2.1. SCOPE

2.1.1 – Seismic loads and analysis requirements to be applied to earthquake resistant design of all cast-in-situ and prefabricated reinforced concrete buildings, structural steel buildings and building-like structures to be built in seismic zones defined in **1.2.3** are specified in this chapter. Rules for masonry buildings are specified in **Chapter 5**.

2.1.2 – Rules for the analysis of building foundations and soil retaining structures are specified in **Chapter 6**.

2.1.3 – Non-building structures which are permitted to be analysed in accordance with the requirements of this chapter shall be limited to those given in **Section 2.12**.

2.1.4 – Analysis rules to be applied to seismic performance assessment and retrofit of existing buildings are given in **Chapter 7**.

2.2. GENERAL GUIDELINES AND RULES

2.2.1. General Guidelines for Building Structural Systems

2.2.1.1 – The building structural system resisting seismic loads as a whole as well as each structural element of the system shall be provided with sufficient stiffness, stability and strength to ensure an uninterrupted and safe transfer of seismic loads down to the foundation soil.

2.2.1.2 – The floor systems should possess sufficient stiffness and strength to ensure the safe transfer of lateral seismic loads between the elements of the structural system. Otherwise appropriate collector elements should be provided.

2.2.1.3 – In order to dissipate a significant part of the seismic energy fed into the structural system, *ductile design* principles specified in **Chapter 3** and in **Chapter 4** of this Specification should be followed.

2.2.1.4 – Design and construction of irregular buildings defined in **2.3.1** should be avoided. Structural system should be arranged symmetrical or nearly symmetrical in plan and torsional irregularity defined as type **A1** irregularity in **Table 2.1** should preferably be avoided. In this respect, it is essential that stiff structural elements such as structural walls should be placed so as to increase the torsional stiffness of the building. On the other hand, vertical irregularities defined as types **B1** and **B2** in **Table 2.1** leading to *weak storey* or *soft storey* at any storey should be avoided.

2.2.1.5 – Effects of rotations of column and in particular wall supporting foundations on soils classified as group (C) and (D) in **Table 6.1** of **Chapter 6** should be taken into account by appropriate methods of structural modelling.

2.2.2. General Rules for Seismic Loads

2.2.2.1 – Unless specified otherwise in this chapter, seismic loads acting on buildings shall be based on *Spectral Acceleration Coefficient* specified in **2.4** and *Seismic Load Reduction Factor* specified in **2.5**.

2.2.2.2 – Unless specified otherwise in this Specification, seismic loads shall be assumed to act non-simultaneously along the two perpendicular axes of the building in the horizontal plane. Rules are given in **2.7.5** for combined effects earthquakes considered.

2.2.2.3 – Unless specified otherwise in this Specification, load factors to be used to determine design internal forces under the combined effects of seismic loads and other loads according to *ultimate strength theory* shall be taken from the relevant structural specifications.

2.2.2.4 – It shall be assumed that the wind loads and seismic loads act non-simultaneously, and the most unfavourable response quantity due to wind or earthquake shall be considered for the design of each structural element. However, even if the quantities due to wind govern, rules given in this Specification shall be applied for dimensioning and detailing of structural elements and their joints.

2.3. IRREGULAR BUILDINGS

2.3.1. Definition of Irregular Buildings

Regarding the definition of irregular buildings whose design and construction should be avoided because of their unfavourable seismic behaviour, types of irregularities in plan and in elevation are given in **Table 2.1** and relevant conditions are given in **2.3.2** below.

2.3.2. Conditions for Irregular Buildings

Conditions related to irregularities defined in **Table 2.1** are given below:

2.3.2.1 – Irregularity types **A1** and **B2** govern the selection of the method of seismic analysis as specified in **2.6** below.

2.3.2.2 – In buildings with irregularity types **A2** and **A3**, it shall be verified by calculation in the first and second seismic zones that the floor systems are capable of safe transfer of seismic loads between vertical structural elements.

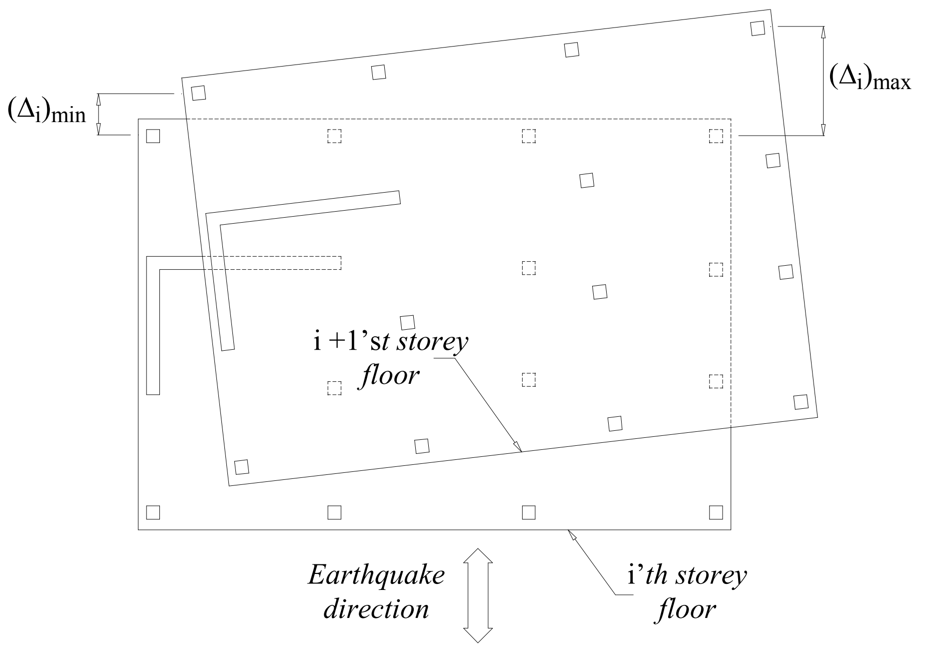
2.3.2.3 – In buildings with irregularity type **B1**, if total infill wall area at i'th storey is greater than that of the storey immediately above, then infill walls shall not be taken into account in the determination of η_{ci} . In the range $0.60 \leq (\eta_{ci})_{min} < 0.80$, Structural Behaviour Factor, given in **Table 2.5** shall be multiplied by $1.25\ (\eta_{ci})_{min}$ which shall be applicable to the entire building in both earthquake directions. In no case, however, $\eta_{ci} < 0.60$ shall be permitted. Otherwise strength and stiffness of the weak storey shall be increased and the seismic analysis shall be repeated.

2.3.2.4 – Conditions related to buildings with irregularities of type **B3** are given below:

(a) In all seismic zones, columns at any storey of the building shall in no case be permitted to rest on the cantilever beams or on top of or at the tip of gussets provided in the columns underneath.

TABLE 2.1 – IRREGULAR BUILDINGS

A – *IRREGULARITIES IN PLAN*	***Related Items***
<u>A1 – Torsional Irregularity :</u> The case where *Torsional Irregularity Factor* η_{bi}, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction, is greater than 1.2 (**Fig. 2.1**). [$\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{ort} > 1.2$] *Storey drifts shall be calculated in accordance with* **2.7**, *by Considering the effects of* ± %5 *additional eccentricities.*	**2.3.2.1**
<u>A2 – Floor Discontinuities :</u> In any floor (**Fig. 2.2**); **I -** The case where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area, **II** – The cases where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements, **III** – The cases of abrupt reductions in the in-plane stiffness and strength of floors.	**2.3.2.2**
<u>A3 – Projections in Plan :</u> The cases where projections beyond the re-entrant corners in both of the two principal directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%. (**Fig. 2.3**).	**2.3.2.2**
B – *IRREGULARITIES IN ELEVATION*	***Related Items***
<u>B1 – Interstorey Strength Irregularity (*Weak Storey*) :</u> In reinforced concrete buildings, the case where in each of the orthogonal earthquake directions, *Strength Irregularity Factor* η_{ci} , which is defined as the ratio of the *effective shear area* of any storey to the *effective shear area* of the storey immediately above, is less than 0.80. [$\eta_{ci} = (\sum A_e)_i / (\sum A_e)_{i+1} < 0.80$] *Definition of effective shear area in any storey* : $\sum A_e = \sum A_w + \sum A_g + 0.15 \sum A_k$ (*See* **3.0** *for notations*)	**2.3.2.2**
<u>B2 – Interstorey Stiffness Irregularity (*Soft Storey*) :</u> The case where in each of the two orthogonal earthquake directions, *Stiffness Irregularity Factor* η_{ki} , which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above or below, is greater than 2.0. [$\eta_{ki} = (\Delta_i/h_i)_{ort} / (\Delta_{i+1}/h_{i+1})_{ort} > 2.0$ or $\eta_{ki} = (\Delta_i /h_i)_{ort} / (\Delta_{i-1}/h_{i-1})_{ort} > 2.0$] *Storey drifts shall be calculated in accordance with* **2.7**, *by considering the effects of* ± %5 *additional eccentricities.*	**2.3.2.1**
<u>B3 - Discontinuity of Vertical Structural Elements :</u> The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath (**Fig. 2.4**).	**2.3.2.4**



In the case where floors behave as rigid diaphragms in their own planes:

$$(\Delta_i)_{ort} = 1/2\ [(\Delta_i)_{max} + (\Delta_i)_{min}]$$

Torsional irregularity factor:

$$\eta_{bi} = (\Delta_i)_{max} \,/\, (\Delta_i)_{ort}$$

Torsional irregularity: $\eta_{bi} > 1.2$

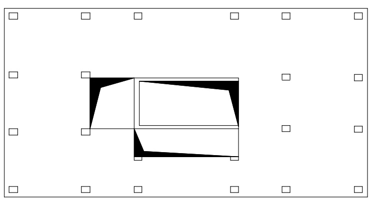
Figure 2.1

$A_b = A_{b1} + A_{b2}$

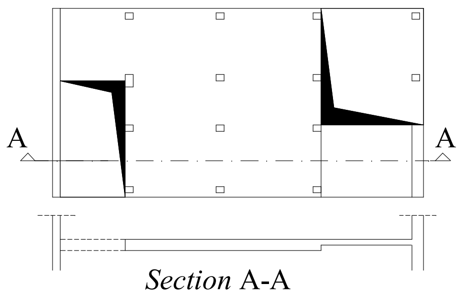
Type **A2** *irregularity* - **I**

$A_b / A > 1/3$

A_b : *Total area of openings*
A : *Gross floor area*

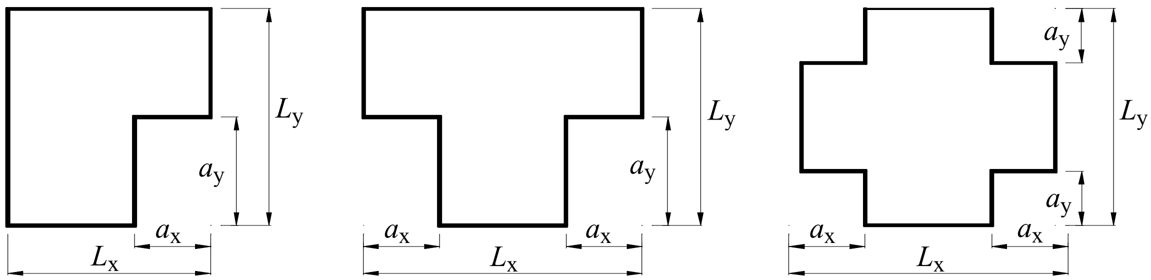


Type **A2** *irregularity* - **II**



Section A-A
Type **A2** *irregularity* - **II** and **III**

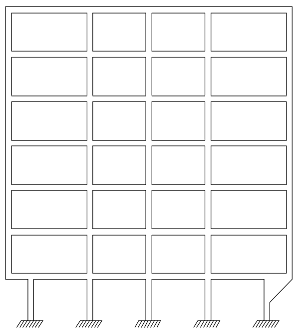
Figure 2.2



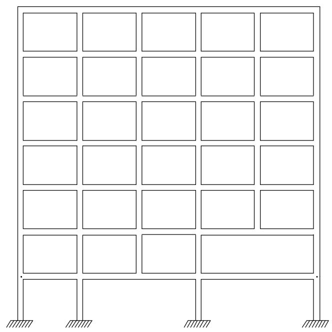
Type **A3** *irregularity :*

$a_x > 0.2\ L_x$ *and at the same time* $a_y > 0.2\ L_y$

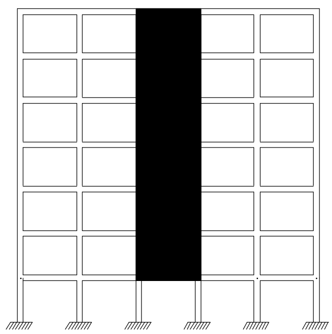
Figure 2.3



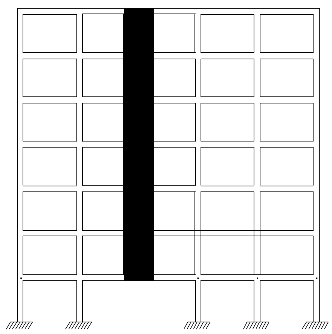
See **2.3.2.4 (a)**



See **2.3.2.4 (b)**



See **2.3.2.4 (c)**



See **2.3.2.4 (d)**

Figure 2.4

(b) In the case where a column rests on a beam which is supported at both ends, all internal force components induced by the combined vertical loads and seismic loads in the earthquake direction considered shall be increased by 50% at all sections of the beam and at all sections of the other beams and columns adjoining to the beam.

(c) In no case the walls shall be permitted to rest on columns underneath.

(d) Structural walls shall in no case be permitted in their own plane to rest on the beam span at any storey of the building.

2.4. DEFINITION OF ELASTIC SEISMIC LOADS: SPECTRAL ACCELERATION COEFFICIENT

The *Spectral Acceleration Coefficient*, $A(T)$, to be considered for determining seismic loads is given by **Eq.(2.1)**. The *elastic spectral acceleration*, $S_{ae}(T)$, which is defined as the ordinate of 5% damped elastic *Design Acceleration Spectrum*, is equal to spectral acceleration coefficient times the acceleration of gravity, g.

$$A(T) = A_o \, I \, S(T)$$
$$S_{ae}(T) = A(T)\, g \qquad \textbf{(2.1)}$$

2.4.1. Effective Ground Acceleration Coefficient

The *Effective Ground Acceleration Coefficient*, A_o , introduced in **Eq.(2.1)** is specified in **Table 2.2**.

TABLE 2.2 – EFFECTIVE GROUND ACCELERATION COEFFICIENT (A_o)

Seismic Zone	A_o
1	0.40
2	0.30
3	0.20
4	0.10

2.4.2. Building Importance Factor

The *Building Importance Factor*, I , given in **Eq.(2.1)** is specified in **Table 2.3**.

TABLE 2.3 – BUILDING IMPORTANCE FACTOR (I)

Purpose of Occupancy or Type of Building	*Importance Factor* (I)
1. Buildings to be utilised after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilised immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods **a)** Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. **b)** Museums	1.4
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	1.2
4. Other buildings Buildings other than above-defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

2.4.3. Spectrum Coefficient

2.4.3.1 – The *Spectrum Coefficient*, $S(T)$, given in **Eq.(2.1)** shall be determined by **Eqs.(2.2)**, depending on local site conditions and the building natural period, T (**Fig.2.5**):

$$
\begin{aligned}
S(T) &= 1 + 1.5\,\frac{T}{T_A} && (0 \le T \le T_A) \\
S(T) &= 2.5 && (T_A < T \le T_B) \qquad \textbf{(2.2)} \\
S(T) &= 2.5\left(\frac{T_B}{T}\right)^{0.8} && (T_B < T)
\end{aligned}
$$

Spectrum Characteristic Periods, T_A and T_B , shown in **Eq.(2.2)** are specified in **Table 2.4**, depending on *Local Site Classes* defined in **Table 6.2** of **Chapter 6**.

TABLE 2.4 – **SPECTRUM CHARACTERISTIC PERIODS** (T_A , T_B)

Local Site Class acc. to Table 12.2	T_A *(second)*	T_B *(second)*
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

2.4.3.2 - In case where the requirements specified in **6.2.1.2** and **6.2.1.3** of **Chapter 6** are not met, spectrum characteristic periods defined in **Table 2.4** for local site class Z4 shall be used.

2.4.4. Special Design Acceleration Spectra

When required, elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However spectral acceleration coefficients corresponding to so obtained acceleration spectrum ordinates shall in no case be less than those determined by **Eq.(2.1)** based on relevant characteristic periods specified in **Table 2.4**.

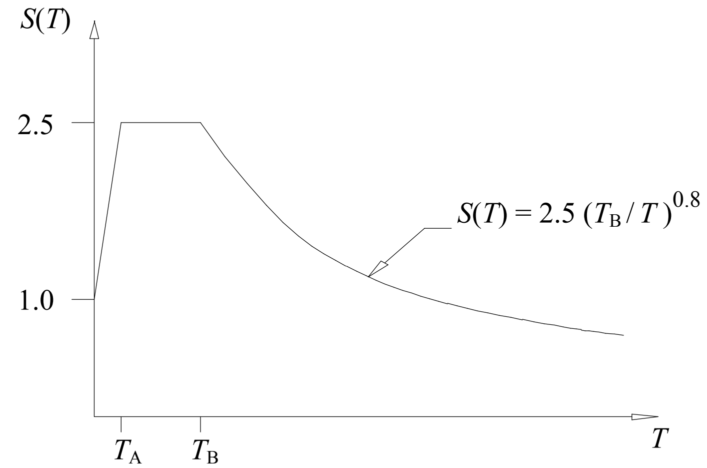


Figure 2.5

2.5. REDUCTION OF ELASTIC SEISMIC LOADS: SEISMIC LOAD REDUCTION FACTOR

Elastic seismic loads determined in terms of *spectral acceleration coefficient* defined in **2.4** shall be divided to below-defined *Seismic Load Reduction Factor* to account for the specific nonlinear behaviour of the structural system during earthquake. *Seismic Load Reduction Factor*, $R_a(T)$, shall be determined by **Eqs.(2.3)** in terms of *Structural Behaviour Factor*, R, defined in **Table 2.5** below for various structural systems, and the natural vibration period T.

$$R_a(T) = 1.5 + (R - 1.5)\,\frac{T}{T_A} \qquad (0 \le T \le T_A)$$
$$R_a(T) = R \qquad (T_A < T) \tag{2.3}$$

2.5.1. General Conditions on Ductility Levels of Structural Systems

2.5.1.1 – Definitions of and requirements to be fulfilled for *structural systems of high ductility level* and *structural systems of nominal ductility level*, for which Structural Behaviour Factors are specified in **Table 2.5**, are given in **Chapter 3** for reinforced concrete buildings and in **Chapter 4** for structural steel buildings.

2.5.1.2 – In structural systems denoted as being *high ductility level* in **Table 2.5**, ductility levels shall be high in both lateral earthquake directions. Systems of high ductility or mixed ductility level in one earthquake direction and of nominal ductility level in the perpendicular earthquake direction shall be deemed to be *structural systems of nominal ductility level* in both directions.

2.5.1.3 – In structural systems where ductility levels are the same in both directions or those with high ductility level in one direction and mixed ductility level in the other direction, different R factors may be used in different directions.

2.5.1.4 – Reinforced concrete flat slab systems without structural walls as well as bare or infilled joist and waffle slab systems whose columns and beams do not satisfy the requirements given in **3.3**, **3.4** and **3.5** shall be treated as *systems of nominal ductility level*.

2.5.1.5 – In the first and second seismic zones;

(a) Excluding paragraph **(b)** below, the use of *structural systems of high ductility level* is mandatory for buildings with structural systems comprised of frames only.

(b) In structural steel buildings with Building Importance Factor to **Table 2.3** is I = 1.2 and I = 1.0, structural systems composed of only frames of nominal ductility level may be used, provided that the condition of $H_N \le 16$ m is met.

(c) In all buildings with Building Importance Factor to **Table 2.3** is I = 1.5 and I = 1.4, *structural systems of high ductility level* or *structural systems of mixed ductility* defined in **2.5.4.1** shall be used.

2.5.1.6 – *Structural systems of nominal ductility level* without structural walls may be permitted only in the third and fourth seismic zones with the following conditions:

(a) Reinforced concrete buildings defined in **2.5.1.4** may be constructed provided that $H_N \le 13$ m.

(b) Excluding those defined in **2.5.1.4**, reinforced concrete and structural steel buildings with structural systems comprised of only frames of nominal ductility level can be constructed provided that $H_N \leq 25$ m.

TABLE 2.5 – STRUCTURAL BEHAVIOUR FACTORS (*R*)

BUILDING STRUCTURAL SYSTEM	***Systems of Nominal Ductility Level***	***Systems of High Ductility Level***
(1) CAST-IN-SITU REINFORCED CONCRETE BUILDINGS		
(1.1) Buildings in which seismic loads are fully resisted by frames..	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls..	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls...	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls...........	4	7
(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS		
(2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	7
(2.2) Buildings in which seismic loads are fully resisted by single-storey frames with columns hinged at top...............	—	3
(2.3) Prefabricated buildings in which seismic loads are fully resisted by prefabricated or cast-in-situ solid and/or coupled structural walls with hinged frame connections	—	5
(2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls	3	6
(3) STRUCTURAL STEEL BUILDINGS		
(3.1) Buildings in which seismic loads are fully resisted by frames..	5	8
(3.2) Buildings in which seismic loads are fully resisted by single-storey frames with columns hinged at top..............	—	4
(3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*......................................	4	5
(b) *Eccentrically braced frames*.......................................	—	7
(c) *Reinforced concrete structural walls*............................	4	6
(3.4) Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*......................................	5	6
(b) *Eccentrically braced frames*.......................................	—	8
(c) *Reinforced concrete structural walls*............................	4	7

2.5.2. Conditions for Solid Structural Wall-Frame Systems of High Ductility Level

Requirements for buildings where seismic loads are jointly resisted by reinforced concrete solid structural walls of *high ductility level* and reinforced concrete or structural steel frames of *high ductility level* are given below:

2.5.2.1 – In order that R=7 can be used for the cases with cast-in-situ reinforced concrete and steel frames or R=6 for the case with prefabricated reinforced concrete frames as given in **Table 2.5**, sum of bending shears developed at the bases of solid structural walls under seismic loads shall not exceed 75% of the total base shear developed for the entire building ($\alpha_S \leq 0.75$).

2.5.2.2 – In the case where the requirement given in **2.5.2.1** cannot be satisfied, R factor to be used in the range $0.75 < \alpha_S \leq 1.0$ shall be determined with $R = 10 - 4\,\alpha_S$ for the cases with cast-in-situ reinforced concrete and steel frames and with $R = 9 - 4\,\alpha_S$ for the case with prefabricated reinforced concrete frames.

2.5.2.3 – In structural walls of $H_w\,/\,\ell_w \leq 2.0$, internal forces calculated according to above-defined R factors shall be amplified by multiplying them with $[3\,/\,(1 + H_w\,/\,\ell_w)]$. However amplification factor shall not be taken more than 2.

2.5.3. Conditions on Mandatory Use of Structural Walls in Certain Systems of Nominal Ductility Level

Structural systems of nominal ductility level defined in paragraphs **(a)** and **(b)** of **2.5.1.6** can also be constructed in all seismic zones as well as above the height limits defined in the same paragraphs. However in such cases it is mandatory to have solid or coupled structural walls of high ductility level or nominal ductility level in full height of reinforced concrete buildings, and concentric or eccentric braced frames of high ductility level or nominal ductility level in structural steel buildings.

2.5.3.1 – In the case where structural walls of *nominal ductility level* are used in the structural system, the sum of shears obtained in each earthquake direction from seismic loads at the bases structural walls shall be more than 75% of the total base shear developed for the entire building.

2.5.3.2 – In the case where structural walls of *high ductility level* are used in the structural system, requirements given below in **2.5.4.1** for mixed structural systems shall be applied.

2.5.4. Conditions for Structural Systems of Mixed Ductility

2.5.4.1 – Structural systems of nominal ductility level defined in paragraphs **(a)** and **(b)** of **2.5.1.6** may be used in combination with structural walls of *high ductility level*. Reinforced concrete solid and coupled structural walls or concentric or eccentric braced frames (for steel buildings) may be used in such structural systems with *mixed systems of ductility levels*, provided that the following conditions are satisfied.

(a) In the analysis of such mixed systems, frames and walls shall be considered jointly, however in all cases $\alpha_S \geq 0.40$ shall be satisfied in each earthquake direction.

(b) When $\alpha_S \geq 2/3$ is satisfied in both earthquake directions, R factor defined in **Table 2.5** for the case where seismic loads are fully resisted by structural walls of *high ductility level* ($R = R_{YP}$) may be used for the entire structural system.

(c) In the range $0.40 < \alpha_S < 2/3$, the relationship of $R = R_{NÇ} + 1.5\ \alpha_S\ (R_{YP} - R_{NÇ})$ shall be applied in both earthquake directions.

2.5.4.2 – Reinforced concrete rigid peripheral walls used in basements of buildings shall not be taken into consideration as parts of structural wall systems or structural wall-frame systems given in **Table 2.5**. Rules to be applied to such buildings are given in **2.7.2.4** and **2.8.3.2**.

2.5.5. Conditions for Buildings with Columns Hinged at Top

2.5.5.1 – In reinforced concrete buildings comprised of single-storey frames with columns hinged at top;

(a) In the case of cast-in-situ reinforced concrete columns, R factor defined for prefabricated buildings in **(2.2)** of **Table 2.5** shall be used.

(b) Requirements applicable to prefabricated reinforced concrete and steel buildings are given in **2.5.5.2**, for which R factors are specified in **(2.2)** and **(3.2)** of **Table 2.5**. The requirements for the use of such frames as the top storey (roof) of cast-in-situ concrete, prefabricated or steel buildings are given in **2.5.5.3**.

2.5.5.2 – A single, partial mezzanine floor can be constructed inside of such single-storey buildings with no more than 25% of plan area of the building. Structural system of mezzanine floor may be taken into account in the seismic analysis together with the main structural frames. In such a case the combined system shall be designed as a system of high ductility level. It shall be checked whether torsional irregularity defined in **Table 2.1** exists in the combined system and if existed it shall be considered in the analysis. The joints of mezzanine floor with the main frames may be hinged or monolithic connection.

2.5.5.3 – In the case where single-storey frames with columns hinged at top are used as the top storey (roof) of cast-in-situ concrete, prefabricated or steel buildings, R factor defined at **(2.2)** or **(3.2)** of **Table 2.5** for top storey, ($R_{üst}$), and the R factor that could be defined differently for the lower stories, (R_{alt}), may be used jointly, provided that the following conditions are met.

(a) Initially seismic analysis shall be performed according to **2.7** or **2.8** with $R = R_{alt}$ considered for the entire building. Reduced and effective story drifts defined in **2.10.1** shall be obtained from this analysis for the entire building.

(b) Internal forces of the top storey shall be obtained by multiplying the internal forces calculated at **(a)** by the ratio ($R_{alt} / R_{üst}$).

(c) Internal forces of the lower stories shall be made of two parts. The first part are those calculated at **(a)**. The second part shall be obtained additionally by applying the forces calculated at **(b)** as support reactions of top storey columns to the structural system of the lower stories after multiplying them by ($1 - R_{üst} / R_{alt}$).

2.6. SELECTION OF ANALYSIS METHOD

2.6.1. Analysis Methods

Methods to be used for the seismic analysis of buildings and building-like structures are, *Equivalent Seismic Load Method* given in **2.7**, *Mode-Combination Method* given in **2.8** and *Analysis Methods in the Time Domain* given in **2.9**. Methods given in **2.8** and **2.9** may be used for the seismic analysis of all buildings and building-like structures.

2.6.2. Application Limits of Equivalent Seismic Load Method

Buildings for which *Equivalent Seismic Load Method* given in **2.7** is applicable are summarised in **Table 2.6**. Methods given in **2.8** or **2.9** shall be used for the seismic analysis of buildings outside the scope of **Table 2.6**.

TABLE 2.6 – BUILDINGS FOR WHICH EQUIVALENT SEISMIC LOAD METHOD IS APPLICABLE

Seismic Zone	*Type of Building*	*Total Height Limit*
1, 2	Buildings with torsional irregularity coefficient satisfying the condition $\eta_{bi} \leq 2.0$ at every storey	$H_N \leq 25$ m
1, 2	Buildings with torsional irregularity coefficient satisfying the condition $\eta_{bi} \leq 2.0$ at every storey and at the same time without type **B2** irregularity	$H_N \leq 40$ m
3, 4	All buildings	$H_N \leq 40$ m

2.7. EQUIVALENT SEISMIC LOAD METHOD

2.7.1. Determination of Total Equivalent Seismic Load

2.7.1.1 – *Total Equivalent Seismic Load* (base shear), V_t , acting on the entire building in the earthquake direction considered shall be determined by **Eq.(2.4)**.

$$V_t = \frac{W A(T_1)}{R_a(T_1)} \geq 0.10\, A_o\, I\, W \tag{2.4}$$

The first natural vibration period of the building, T_1 , shall be calculated in accordance with **2.7.4**.

2.7.1.2 – Total building weight, W, to be used in **Eq.(2.4)** as the seismic weight shall be determined by **Eq.(2.5)**.

$$W = \sum_{i=1}^{N} w_i \tag{2.5}$$

Storey weights w_i of **Eq.(2.5)** shall be calculated by **Eq.(2.6)**.

$$w_i = g_i + n\, q_i \tag{2.6}$$

Live Load Participation Factor, n , shown in **Eq.(2.6)** is given in **Table 2.7**. In industrial buildings, n = 1 shall be taken for fixed equipment weights while crane payloads shall not be taken into account in the calculation of storey weights. In the calculation of roof weights for seismic loads, 30% of snow loads shall be considered.

TABLE 2.7 – LIVE LOAD PARTICIPATION FACTOR (n)

Purpose of Occupancy of Building	n
Depot, warehouse, etc.	0.80
School, dormitory, sport facility, cinema, theatre, concert hall, car park, restaurant, shop, etc.	0.60
Residence, office, hotel, hospital, etc.	0.30

2.7.2. Determination of Design Seismic Loads Acting at Storey Levels

2.7.2.1 – Total equivalent seismic load determined by **Eq.(2.4)** is expressed by **Eq. (2.7)** as the sum of equivalent seismic loads acting at storey levels (**Fig. 2.6a**):

$$V_t = \Delta F_N + \sum_{i=1}^{N} F_i \qquad \textbf{(2.7)}$$

2.7.2.2 – The value of *additional equivalent seismic load*, ΔF_N, acting at the N'th storey (roof) of the building shall be determined by **Eq.(2.8)**.

$$\Delta F_N = 0.0075\, N\, V_t \qquad \textbf{(2.8)}$$

2.7.2.3 – Excluding ΔF_N, the remaining part of the total equivalent seismic load shall be distributed to stories of the building (including N'th storey) in accordance with **Eq.(2.9)**.

$$F_i = (V_t - \Delta F_N)\, \frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j} \qquad \textbf{(2.9)}$$

2.7.2.4 – In buildings with reinforced concrete peripheral walls at their basements being very rigid relative to upper stories and basement floors behaving as rigid diaphragms in horizontal planes, equivalent seismic loads acting on the basement stories and on the upper stories shall be calculated independently as in the following. These loads shall be applied together to the combined structural system.

(a) In determining the total equivalent seismic load and equivalent storey seismic loads in accordance with **2.7.1.1**, **2.7.2.2** and **2.7.2.3**, appropriate R factor shall be selected from **Table 2.5** without considering the rigid peripheral basement walls and seismic weights of the upper stories only shall be taken into account. In this case, foundation top level considered in the relevant definitions and expressions shall be replaced by the ground floor level. Fictitious loads used for the calculation of the first natural vibration period in accordance with **2.7.4.1** shall also be based on seismic weights of the upper stories only (**Fig.2.6b**).

(b) In calculating equivalent seismic loads acting on rigid basement stories, seismic weights of basements only shall be taken into account and calculation shall be independent of upper stories. For such parts of the building, *Spectrum Coefficient* shall be taken as $S(T) = 1$ without calculating the natural vibration period. In determining equivalent seismic loads acting on each basement storey, spectral acceleration obtained from **Eq.(2.1)** shall be multiplied directly with the respective weight of the storey and resulting elastic loads shall be reduced by dividing them to $R_a(T) = 1.5$ (**Fig.2.6c**).

(c) In-plane strength of ground floor system, which is surrounded by very stiff basement walls and located in the transition zone between upper stories, shall be checked according to the internal forces obtained from this analysis.

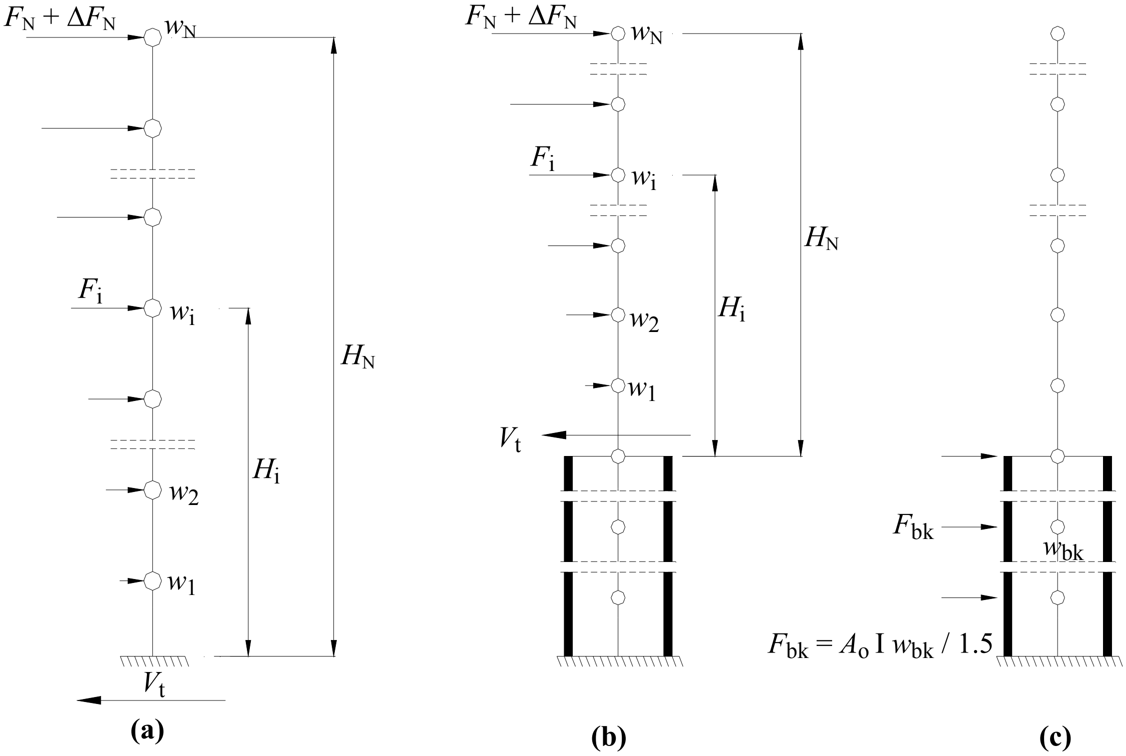


Figure 2.6

2.7.3. Displacement Components to be Considered and Application Points of Seismic Loads

2.7.3.1 – In buildings where floors behave as rigid horizontal diaphragms, two lateral displacement components and the rotation around the vertical axis shall be taken into account at each floor as independent static displacement components. At each floor, equivalent seismic loads determined in accordance with **2.7.2** shall be applied to the floor mass centre as well as to the points defined by shifting it +5% and −5% of the floor length in the perpendicular direction to the earthquake direction considered in order to account for the *additional eccentricity effects* (**Fig. 2.7**).

2.7.3.2 – In buildings where type **A2** irregularity exists and floors do not behave as rigid horizontal diaphragms, sufficient number of independent static displacement components shall be considered to account for the in-plane deformation of floors. In order to consider additional eccentricity effects, each of the seismic loads acting on the individual masses distributed over each floor shall be shifted by +5% and −5% of the floor length in perpendicular direction to the earthquake direction considered (**Fig. 2.8**).

2.7.3.3 – In the case where type **A1** irregularity defined in **Table 2.1** exists at any i'th storey such that the condition $1.2 < \eta_{bi} \leq 2.0$ is satisfied, ±5% additional eccentricity applied to this floor according to **2.7.3.1** and/or **2.7.3.2** shall be amplified by multiplying with coefficient D_i given by **Eq.(2.10)** for both earthquake directions.

$$D_i = \left(\frac{\eta_{bi}}{1.2} \right)^2 \qquad \textbf{(2.10)}$$

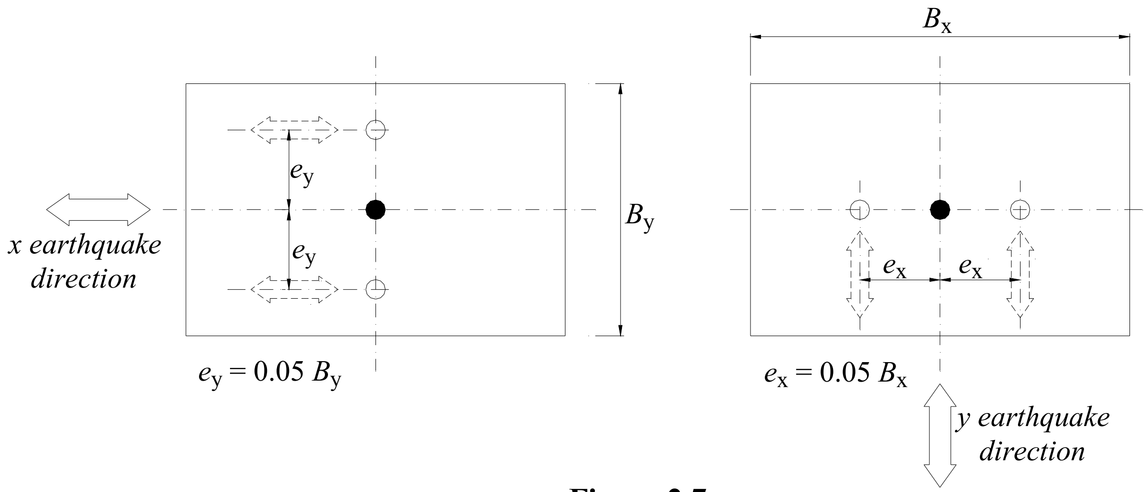


Figure 2.7

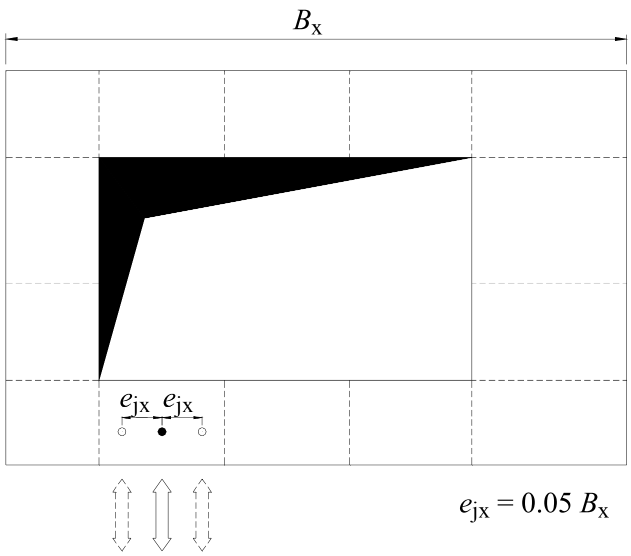


Figure 2.8

2.7.4. Determination of First Natural Vibration Period of Building

2.7.4.1 – In the case where Equivalent Seismic Load Method is applied, the natural vibration period of the building dominant in the earthquake direction shall not be taken longer than the value calculated by **Eq.(2.11)**.

$$T_1 = 2\pi \left(\frac{\sum_{i=1}^{N} m_i\, d_{fi}^2}{\sum_{i=1}^{N} F_{fi}\, d_{fi}} \right)^{1/2} \qquad \textbf{(2.11)}$$

Fictitious load F_{fi} acting on the i'th storey shall be obtained from **Eq.(2.9)** by substituting any value (*for example a unit value*) in place of $(V_t - \Delta F_N)$, see **Fig. 2.9**.

2.7.4.2 – Regardless of the value calculated by **Eq.(2.11)**, natural period shall not be taken longer than $0.1N$ in buildings with $N > 13$ excluding basement(s).

$$F_{fi} = \frac{w_i\, H_i}{\sum_{j=1}^{N} w_j\, H_j}$$

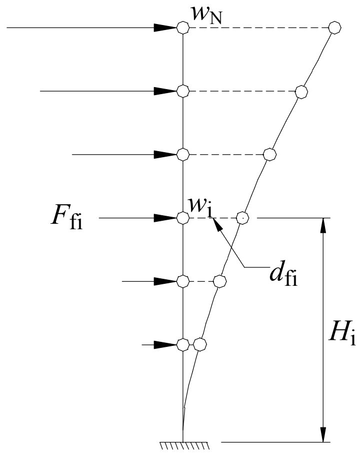


Figure 2.9

2.7.5. Internal forces in Element Principal Axes

Under the combined effects of independently acting x and y direction earthquakes to the structural system, internal forces in element principal axes a and b shall be obtained by **Eq.(2.12)** such that the most unfavourable results yield **(Fig. 2.10)**.

$$B_a = \pm B_{ax} \pm 0.30\, B_{ay} \quad \text{veya} \quad B_a = \pm 0.30\, B_{ax} \pm B_{ay}$$
$$B_b = \pm B_{bx} \pm 0.30\, B_{by} \quad \text{veya} \quad B_b = \pm 0.30\, B_{bx} \pm B_{by} \qquad \textbf{(2.12)}$$

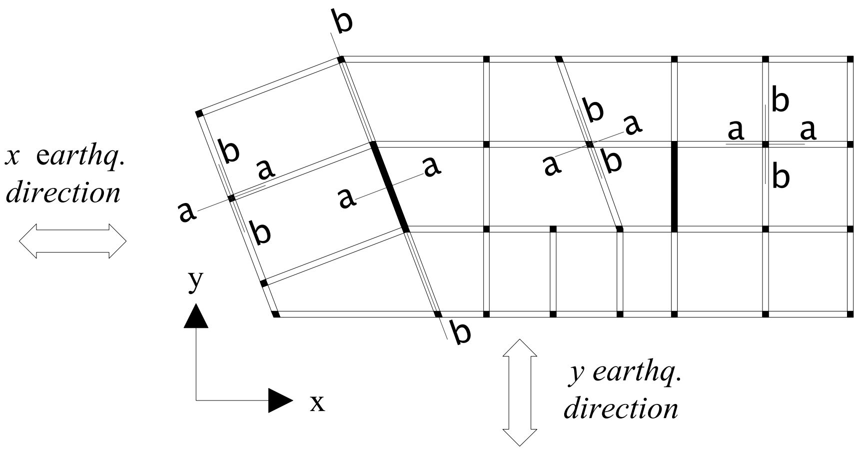


Figure 2.10

2.8. MODE COMBINATION METHOD

In this method, maximum internal forces and displacements are determined by the statistical combination of maximum contributions obtained from each of the sufficient number of natural vibration modes considered.

2.8.1. Acceleration Spectrum

Reduced acceleration spectrum ordinate to be taken into account in any n'th vibration mode shall be determined by **Eq.(2.13)**.

$$S_{aR}(T_n) = \frac{S_{ae}(T_n)}{R_a(T_n)} \qquad \textbf{(2.13)}$$

In the case where elastic design acceleration spectrum is determined through special investigations in accordance with **2.4.4**, relevant spectrum ordinate shall be considered in **Eq.(2.13)** in lieu of $S_{ae}(T_n)$.

2.8.2. Dynamic Degrees of Freedom to be Considered

2.8.2.1 – In buildings where floors behave as rigid horizontal diaphragms, two horizontal degrees of freedom in perpendicular directions and a rotational degree of freedom with respect to the vertical axis passing through mass centre shall be considered at each storey. At each floor, modal seismic loads shall be determined for those degrees of freedom and shall be applied to the floor mass centre as well as to the points defined by shifting it +5% and −5% of the floor length in the perpendicular direction to the earthquake direction considered in order to account for the *additional eccentricity effects* (**Fig. 2.7**).

2.8.2.2 – In buildings where type **A2** irregularity exists and floors do not behave as rigid horizontal diaphragms, sufficient number of dynamic degrees of freedom shall be considered to account for the in-plane deformation of floors. In order to consider additional eccentricity effects, each of the modal seismic loads acting on the individual masses distributed over each floor shall be shifted by +5% and −5% of the floor length in perpendicular direction to the earthquake direction considered (**Fig. 2.8**). In such buildings, internal force and displacement quantities due to additional eccentricity effects alone may also be calculated in accordance with **2.7**. Such quantities shall be directly added to those combined in accordance with below given **2.8.4** without taking into account additional eccentricity effects.

2.8.3. Sufficient Number of Vibration Modes to be Considered

2.8.3.1 – *Sufficient number of vibration modes*, Y, to be taken into account in the analysis shall be determined to the criterion that the sum of effective participating masses calculated for each mode in each of the given x and y perpendicular lateral earthquake directions shall in no case be less than 90% of the total building mass.

$$\sum_{n=1}^{Y} M_{xn} = \sum_{n=1}^{Y} \frac{L_{xn}^2}{M_n} \geq 0.90 \sum_{i=1}^{N} m_i$$
$$\sum_{n=1}^{Y} M_{yn} = \sum_{n=1}^{Y} \frac{L_{yn}^2}{M_n} \geq 0.90 \sum_{i=1}^{N} m_i \qquad \textbf{(2.14)}$$

The expressions of L_{xn} , L_{yn} and modal mass M_n shown in **Eqs.(2.14)** are given below for buildings with rigid floor diaphragms:

$$L_{xn} = \sum_{i=1}^{N} m_i \Phi_{xin} \quad ; \quad L_{yn} = \sum_{i=1}^{N} m_i \Phi_{yin}$$
$$M_n = \sum_{i=1}^{N} (m_i \Phi_{xin}^2 + m_i \Phi_{yin}^2 + m_{\theta i} \Phi_{\theta in}^2) \qquad \textbf{(2.15)}$$

2.8.3.2 – In buildings with reinforced concrete peripheral walls at their basements being very rigid relative to upper stories and basement floors behaving as rigid diaphragms in horizontal planes, it may be sufficed with the consideration of vibration modes which are effective in the upper stories only. In this case, in the analysis performed by the Mode Combination Method which corresponds to the analysis by Equivalent Seismic Load Method as given in Paragraph **(a)** of **2.7.2.4**, the coefficient R shall be selected from **Table 2.5** without considering the rigid peripheral basement walls whereas the upper storey masses only shall be taken into account. Paragraphs **(b)** and **(c)** of **2.7.2.4** shall be applied as they are given for Equivalent Seismic Load Method.

2.8.4. Modal Combination

Rules to be applied for the statistical combination of non-simultaneous maximum contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacements and storey drifts, are specified in the following *provided that they are applied independently for each response quantity*:

2.8.4.1 – In the cases where natural periods of any two vibration mode with $T_m < T_n$ always satisfy the condition $T_m / T_n < 0.80$, *Square Root of Sum of Squares (SRSS) Rule* may be applied for the combination of maximum modal contributions.

2.8.4.2 – In the cases where the above given condition is not satisfied, *Complete Quadratic Combination (CQC) Rule* shall be applied for the combination of maximum modal contributions. In the calculation of *cross correlation coefficients* to be used in the application of the rule, modal damping factors shall be taken as 5% for all modes.

2.8.5. Lower Limits of Response Quantities

In the case where the ratio of the base shear in the given earthquake direction, V_{tB}, which is obtained through modal combination according to **2.8.4**, to the base shear, V_t, obtained by Equivalent Seismic Load Method through **Eq.2.4** is less than the below given value of β ($V_{tB} < \beta V_t$), all internal force and displacement quantities determined by Mode Combination Method shall be amplified in accordance with **Eq.(2.16)**.

$$B_D = \frac{\beta V_t}{V_{tB}} B_B \qquad \textbf{(2.16)}$$

If at least one of the irregularities of type **A1**, **B2** or **B3** defined in **Table 2.1** exists in a building β=0.90, whereas none of them exists β=0.80 shall be used in **Eq. (2.16)**.

2.8.6. Internal forces in Element Principal Axes

Under the combined effects of independently acting x and y direction earthquakes to the structural system, the directional combination rule given in **2.7.5** shall be additionally applied to the internal forces obtained in element principal axes a and b by modal combination according to **2.8.4** – see **Fig. 2.10**.

2.9. ANALYSIS METHODS IN TIME DOMAIN

Artificially generated, previously recorded or simulated earthquake ground motions can be used in linear or nonlinear seismic analysis of buildings and building-like structures in the time domain.

2.9.1. Artificial Earthquake Ground Motions

In the case where artificial ground motions are used, at least three earthquake ground motions shall be generated with the following properties.

(a) The duration of the strong motion part shall neither be shorter than 5 times the fundamental period of the building nor 15 seconds.

(b) Mean spectral acceleration of generated ground motions for zero period shall not be less than $A_o g$.

(c) Mean spectral accelerations of artificially generated acceleration records for 5% damping ratio shall not be less than 90% of the elastic spectral accelerations, $S_{ae}(T)$, defined in **2.4** in the period range between $0.2T_1$ and $2T_1$ with respect to dominant natural period, T_1 , of the building in the earthquake direction considered. In the case where *linear elastic analysis* is performed in the time domain, spectral accelerations defined by **Eq.(2.13)** shall be considered to define the reduced earthquake ground motion.

2.9.2. Recorded or Simulated Earthquake Ground Motions

Recorded earthquakes or physically simulated ground motions with appropriate source and wave propagation characteristics can be used for seismic analysis to be performed in the time domain. Local site conditions should be appropriately considered in selecting or generating such ground motions. At least three earthquake ground motions shall be selected or generated satisfying all of the conditions given in **2.9.1**.

2.9.3. Analysis in the Time Domain

In the case where nonlinear analysis is performed in the time domain, internal force-deformation relationships representing the dynamic behaviour of elements of structural system under cyclic loads shall be defined through relevant literature with proven theoretical and experimental validations. If three ground motions are used the maxima of the results, and if at least seven ground motions are used the mean values of the results shall be considered for design.

2.10. LIMITATION OF DISPLACEMENTS, SECOND ORDER EFFECTS AND SEISMIC JOINTS

2.10.1. Calculation and Limitation of Effective Storey Drifts

2.10.1.1 – The *reduced storey drift*, Δ_i , of any column or structural wall shall be determined by **Eq.(2.17)** as the difference of displacements between the two consecutive stories.

$$\Delta_i = d_i - d_{i-1} \qquad \textbf{(2.17)}$$

In **Eq.(2.17)** d_i and d_{i-1} represent lateral displacements obtained from the analysis at the ends of any column or structural wall at stories i and (i - 1) under reduced seismic loads. However the condition given in **2.7.4.2** as well as the minimum equivalent seismic load condition defined by **Eq.(2.4)** may not be considered in the calculation of d_i and Δ_i.

2.10.1.2 – Effective storey drift , δ_i , of columns or structural walls at the i'th storey of a building shall be obtained for each earthquake direction by **Eq.(2.18)**.

$$\delta_i = R\ \Delta_i \tag{2.18}$$

2.10.1.3 – The maximum value of effective storey drifts, $(\delta_i)_{max}$, obtained for each earthquake direction by **Eq.(2.18)** at columns or structural walls of a given i'th storey of a building shall satisfy the condition given by **Eq.(2.19)**:

$$\frac{(\delta_i)_{max}}{h_i} \leq 0.02 \tag{2.19}$$

This limit may be exceeded by 50% in single storey frames where seismic loads are fully resisted by steel frames with joints capable of transferring cyclic moments.

2.10.1.4 – In the case where the condition given by **Eq.(2.19)** is not satisfied at any storey of the building, the seismic analysis shall be repeated with increased stiffness of the structural system. However, even if the condition is satisfied, serviceability of non-structural brittle elements (e.g. façade elements) under effective storey drifts shall be verified by calculation.

2.10.2. Second-Order Effects

Unless a more refined analysis considering the nonlinear behaviour of structural system is performed, second-order effects may be taken into account in accordance with **2.10.2.1**.

2.10.2.1 – In the case where *Second-Order Effect Indicator*, θ_i , satisfies the condition given by **Eq.(2.20)** for the earthquake direction considered at each storey, second-order effects shall be evaluated in accordance with the currently enforced specifications of reinforced concrete or structural steel design.

$$\theta_i = \frac{(\Delta_i)_{ort} \sum\limits_{j=i}^{N} w_j}{V_i\, h_i} \leq 0.12 \tag{2.20}$$

$(\Delta_i)_{ort}$ shall be determined in accordance with **2.10.1.1** as the average value of reduced storey drifts calculated for i'th storey columns and structural walls.

2.10.2.2 – In the case where the condition given by **Eq.(2.20)** is not satisfied, seismic analysis shall be repeated with sufficiently increased stiffness of the structural system.

2.10.3. Seismic Joints

Excluding the effects of differential settlements and rotations of foundations and the effects of temperature change, sizes of gaps to be retained in the seismic joints between building blocks or between the old and newly constructed buildings shall be determined in accordance with the following conditions:

2.10.3.1 – Unless a larger value is obtained in accordance with **2.10.3.2** below, sizes of gaps shall not be less than the square root of sum of squares of average storey displacements multiplied by the coefficient α specified below. Storey displacements to be considered are the average values of reduced displacements d_i calculated within a storey at the column or structural wall joints. In the cases where the seismic analysis is not performed for the existing old building, the storey displacements shall not be assumed to be less than those obtained for the new building at the same stories.

(a) $\alpha = R\ /\ 4$ shall be taken if all floor levels of adjacent buildings or building blocks are the same.

(b) $\alpha = R\ /\ 2$ shall be taken if any of the floor levels of adjacent buildings or building blocks are not the same.

2.10.3.2 – Minimum size of gaps shall be 30 mm up to 6 m height. From thereon a minimum 10 mm shall be added for every 3 m height increment.

2.10.3.3 – Seismic joints shall be arranged to allow the independent movement of building blocks in all earthquake directions.

2.11. SEISMIC LOADS APPLIED TO STRUCTURAL APPENDAGES, ARCHITECTURAL ELEMENTS, MECHANICAL AND ELECTRICAL EQUIPMENT

2.11.1 – Equivalent seismic loads to be applied to structural appendages such as balconies, parapets, chimneys, etc. and to all architectural elements such as façade and partition panels, etc. as well as the seismic loads to be used for the connections of mechanical and electrical equipment to the structural system elements are given by **Eq.(2.21)**.

$$f_e = 0.5\, A_o\, I\, w_e \left(1 + 2\frac{H_i}{H_N}\right) \qquad \textbf{(2.21)}$$

The seismic load shall be applied horizontally to the mass centre of the element concerned in a direction to result in most unfavourable internal forces. The seismic loads to be applied to non-vertical elements shall be half the equivalent seismic load calculated by **Eq.(2.21)**.

2.11.2 – In the case where the sum of mechanical or electrical equipment weights, as denoted by w_e in **Eq.(2.21)**, exceeds $0.2w_i$ at any i'th storey, equipment weights and the stiffness properties of their connections to the building shall be taken into account in the earthquake analysis of the building structural system.

2.11.3 – In the case where *floor acceleration spectrum* is determined by appropriate methods to define the peak acceleration at the floor where mechanical or electrical equipment is located, **Eq.(2.21)** may not be applied.

2.11.4 – Twice the seismic load calculated by **Eq.(2.21)** or determined according to **2.11.3** shall be considered for fire extinguishing systems, emergency electrical systems as well as for equipments connecting to infill walls and for their connections

2.12. NON-BUILDING STRUCTURES

Non-building structures permitted to be analysed in accordance with the requirements of this chapter and the corresponding *Structural Behaviour Factors*, (*R*), to be applied to such structures are given in **Table 2.8**. Applicable *Seismic Load Reduction Factors* shall be determined in accordance with **Eq.(2.3)**. Where applicable, *Building Importance Factors* specified in **Table 2.3** shall be used for non-building structures. However *Live Load Participation Factors* specified in **Table 2.7** shall not be applied. Except snow loads and crane payloads, unreduced weights of all solid and liquid materials stored and mechanical equipment shall be used.

TABLE 2.8 - STRUCTURAL BEHAVIOUR FACTORS FOR NON-BUILDING STRUCTURES

TYPE OF STRUCTURE	*R*
Elevated liquid tanks, pressurised tanks, bunkers, vessels carried by frames of high ductility level or steel eccentric braced frames	4
Elevated liquid tanks, pressurised tanks, bunkers, vessels carried by frames of nominal ductility level or steel concentric braced frames	2
Cast-in-situ reinforced concrete silos and industrial chimneys with uniformly distributed mass along height (*)	3
Reinforced concrete cooling towers (*)	3
Space truss steel towers, steel silos and industrial chimneys with uniformly distributed mass along height (*)	4
Guyed steel high posts and guyed steel chimneys	2
Inverted pendulum type structures carried by a single structural element with mass concentrated at the top	2
Industrial type steel storage racks	4

(*) *Analysis of such structures shall be performed in accordance with* **2.8** *or* **2.9** *by considering sufficient number of discrete masses defined along the structure.*

2.13. REQUIREMENTS FOR SEISMIC ANALYSIS REPORTS

The following requirements shall apply to the analysis reports that include seismic analysis of buildings:

2.13.1 - Types of irregularities specified in **Table 2.1** shall be evaluated in detail for the building to be designed and, if any, existing irregularities shall be identified.

2.13.2 - The selected structural system of high or nominal ductility level shall be clearly defined with respect to the requirements of **Chapter 3** or **Chapter 4**, and the selection of the applicable *R* factor from **Table 2.5** shall be explained.

2.13.3 - The selection of the applicable analysis method in accordance with **2.6** shall be clearly explained by considering the seismic zone, building height and structural irregularities involved.

2.13.4 - The following rules shall be applied in the cases where the analysis is performed by computer:

(a) Analysis report shall include three-dimensional illustrations of structural system by indicating the joint and element numbering.

(b) All input data as well as output data including internal forces and displacements shall be included in the analysis report in an easily understandable format.

(c) The title, author and the version of the computer software used in the analysis shall be clearly indicated.

(d) When requested by the approval authority, theory manual and user's guide of the computer software shall be included in the analysis report.

2.14. INSTALLATION OF STRONG MOTION RECORDERS

Upon endorsement by the Ministry of Public Works and Settlement, strong ground motion accelographs shall be permitted to be installed by the ministry or university institutions on the public, private or corporate buildings and other structures for the purpose of recording the strong earthquake motions, and owners or operators of buildings or structures shall be responsible from the safety of such instruments.

Appendix H3

Structural Design Report and Calculations

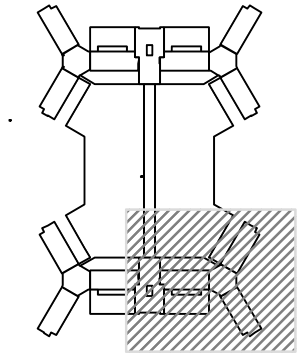
PROJE ADI:
PROJECT TITLE:

GAZİANTEP ENTEGRE SAĞLIK KAMPÜSÜ
GAZIANTEP INTEGRATED HEALTH CAMPUS

PROJE KODU:
PROJECT CODE:

GESK

ANAHTAR PLAN:
KEY PLAN:



İDARE:
CLIENT:

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SAĞLIK YATIRIMLARI GENEL MÜDÜRLÜĞÜ
KAMU ÖZEL İŞBİRLİĞİ DAİRE BAŞKANLIĞI

T.C. Sağlık Bakanlığı

Kamu Özel Ortaklığı Daire Başkanlığı
Department of Public Private Partnership

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INSPECTOR:

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MÜŞAVİR:
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ÇİZİM ADI:
DRAWING TITLE:
ANA HASTANE BİNASI HESAP RAPORLARI
MAIN HOSPITAL BUILDING CALCULATION REPORTS

ST

PAFTA ADI:
SHEET NAME:

DWG ADI:
DWG NAME:

ÖLÇEK:
SCALE:

TARİH:
DATE:

KAT ALANI:
FLOOR AREA:

INDEX

PURPOSE

This report describes aspects of the structural design approach for Gaziantep Integrated Health Campus . Projects prepared without the comments of the other disciplines. Responsibilty belongs to the investor.

SCOPE

This report is applicable to the structural design of Gaziantep Integrated Health Campus only.

1 INTRODUCTION

Gaziantep Integrated Health Campus Structural Calculations and Design Report defines the structural system of the building and provides a brief of the general design approach to be followed for the building. The objective of this document is to address the main structural decisions for the building, proposed materials, reference documents taken as basis and the software used.

1.1 CODES & STANDARTS

The codes and standards used in the structural design of the building are listed below.

TS 500 :Requirements for Design and Construction of Reinforced Concrete Structures

TS 498 :Design Loads for Buildings

SFBBISZ 2007 :Specifications for Structurs to be Built in Disaster Areas 2007

1.2 UNIT SYSTEM

This report uses International System of Units (SI).

Length : m, cm, mm

Force : ton-force (Tonf)

Stress : tf / m^2

Moment : tf-m

Unit Weight : tf / m^3

Mass : ton

1.3 MATERIAL PROPERTIES

Lean Concrete : C12 f_{ck} = 120 kg / cm², f_{cd} = 80 kg / cm²

Structural Concrete : C25 f_{ck} = 250 kg / cm², f_{cd} = 167 kg / cm² (for foundations)

Structural Concrete : C30 f_{ck} = 300 kg / cm², f_{cd} = 200 kg / cm² (for structural elements)

Reinforcement : S420 f_{yk} = 4200 kg / cm², f_{yd} = 3650 kg / cm²

1.4 CONCRETE COVER (CLEAR COVER)

Concrete cover is defined as the clear distance from the concrete surface to the outermost surface of the steel to which the cover requirement applies. It is measured to the outer edge of stirrups, ties or spirals if transverse reinforcement encloses main bars and to the outermost layer of bars if more than one layer is used without stirrups or ties. Unless otherwise indicated, concrete cover over reinforcement will be as follows,

Foundations and other buried elements : 50 mm

Superstructure elements: columns : 40 mm

Superstructure elements: beams : 40 mm

Superstructure elements: slabs : 25 mm

Superstructure elements: walls : 40 mm

According to “Fire Safety of Buildings 2007” codes Section 23; fire requirement refers to 120 minute.

1.5 SOFTWARE

- STA4-CAD Structural Analysis For Computer Aided Design
- AutoCAD 2010, Computer Aided Drafting, Autodesk Inc.

1.6 UNIT WEIGHTS

Self weights of the structural members are calculated and taken into account by the analysis & design software automatically during analysis. Other loadings acting on the structure are shown in details in the following chapters of this report. Material unit weights to be used are as follows,

Reinforced concrete : 2.5 t / m³

Compacted backfill : 1.9 t / m³, Ø=30°

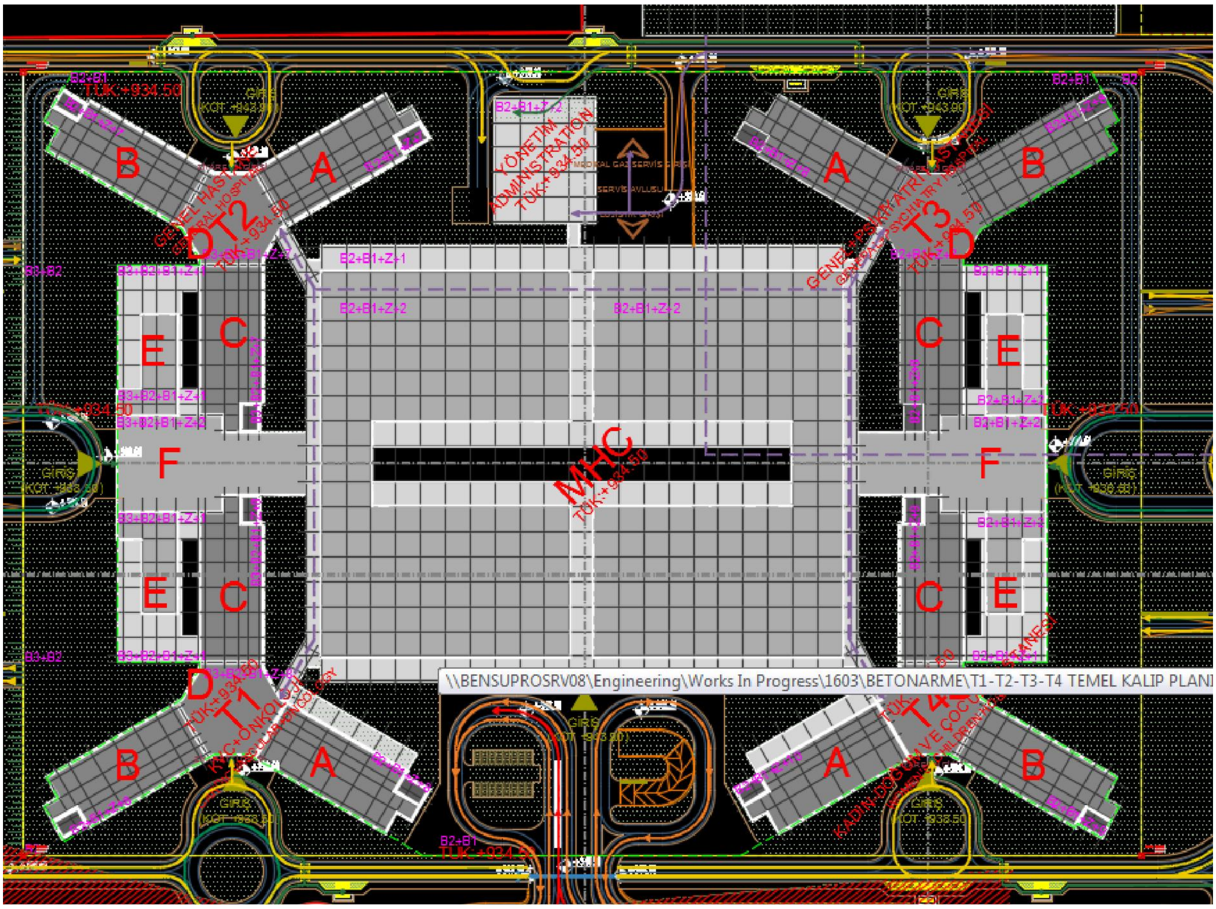
2 DESIGN CONDITIONS

2.1 DESIGN PRINCIPLES

All of the loads and load effects are taken into account that the structural system of the building is going to withstand during its service lifetime. Structural design is based on strength, functionality, serviceability, aesthetic, economical and constructability (practical) considerations.

2.2 GENERAL BUILDING INFORMATION

General architectural for of building has got different structural shapes. For that reason, building dilated different parts. Additionally, all information about dilated blocks floor counts and measurement details given in the analysis and design section. Detailed key plan given below. The buildings planned as hospitals.



2.3 ANALYTICAL MODEL

The building is modeled in 3-Dimensions (3-D) using the software described in Section 3.5, reflecting the correct behavior of the structure. All of the material properties, load effects that the structure can encounter during its service lifetime and limit states are defined in the model. 3-D model was set to analyze and design all structural members as required by the codes and standards described in Section 3.1. General properties of the analytical models used are as follows,

a) 6 degrees of freedom was used at every joint point.
b) Beam and concrete structural members are modeled as frame elements.
c) Shear walls are modeled as panel elements.
   The foundation and the flat slabs are modeled as shell element.
d) All of the material properties, load effects that the structure can encounter during its service lifetime and limit states are defined in the model.
e) For modelling cooperation of building and foundation, necessary information is taken from the soil investigation report and all building and foundation calculations completed with the soil spring values.
f) Detailed analysis and design informations given at the next stages of this report.

2.4 STRUCTURAL SYSTEM INFORMATION

** has been designed as reinforced concrete building formed of columns and structural walls and flat slabs connecting each other over raft foundation supports.*

** Building and building members are designed to provide design strength at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in related codes indicated below. All members also are met all other requirements at service load level to ensure adequate performance.*

Turkish EQ Code 2007 Section 3.3.1.1 and TS 500 Section 7.4.1;

- *Shorter dimension of columns with rectangular section shall not be less than 250 mm and section area shall not be less than 75000 mm2. Minimum column dimension is 400 mm and minimum section area 320000 mm2 in our building. (Minimum column dimensions are 400 mm and 800 mm).*

TS 500 Section 11.4.2;

- *Flat slabs are designed without beams, which are two way slab. Minimum thickness of slab is calculated as per to TS 500 Part 11.4.2. Minimum thickness is 1/30 of the span length. So thickness requirement for 840 cm span slab; 840/30 = 28 cm. Chosen slab thickness is 30 cm.*

TS 500 Section 11.4.2;

- *Two way slab thickness is calculated with the TS500 Section 11.4.2 equation 11.1. Minimum slab thickness calculated for two way slab with the beams and minimum thickness is 21cm. Minimum slab thickness 25cm and it's appropriate with regulations.*

Turkish EQ Code 2007 Section 3.4.1.1 and TS 500 Section 7.3;

- *Width of the beam shall be at least 250 mm.*
- *The total depth of a beam shall neither be less then 300 mm nor less then three times the slab thickness. Beam height shall not be less than 3 times the thickness of floor slab and 300 mm, nor shall it more than 3.5 times the beam web width. In our buildings, minimum beam section is 30/90 cm and these measurements appropriate with regulations.*

Turkish EQ Code 2007 Section 3.4.1.1 and TS 500 Section 7.3;

- *Width of the beam shall be at least 250 mm.*
- *The total depth of a beam shall neither be less then 300 mm nor less then three times the slab thickness. Beam height shall not be less than 3 times the thickness of floor slab and 300 mm, nor shall it more than 3.5 times the beam web width. In our buildings, minimum beam section is 40/75 cm and these dimensions appropriate with regulations.*

Turkish EQ Code 2007 Section 3.6.1;

- *Shear wall web section thickness provides with the regulation of "wall thickness shall not be less than 1 / 15 the storey height and 200 mm". Shear wall thickness is 400 mm and maximum story height is 6000 mm. According to Turkish EQ Code 2007 regulation, 6000/15=400 mm is appropriate.*

1.2.a. *Beams*

Beams are used for secondary purpose as shown below in all blocks to contact slabs at different levels. Beam calculation reports are shown in calculation report appendixes.

Generally inner beams dimension is 50cm/75cm and outer beams dimension is 40cm/75cm.

The height of beam 90 cm is designed where fire trucks are passing.

1.2.b. Slabs

Flat slab is designed without beams. General slab thickness is 30 cm. w Slab thickness is calculated as per to TS500 section 11.4.2

The depth of a two way slab without beams (flat plate or flat slab) cannot be less than the values given below:

Slabs without beams and drop panels	$h \geq \ell_n/30$ and $h \geq 180$ mm
Slabs with drop panels but without beams	$h \geq \ell_n/35$ and $h \geq 140$ mm

Also slabs are designed with column & middle strips in STA4Cad programme.

Considering of punching shear reinforcement section in TS500 (8.3.2);

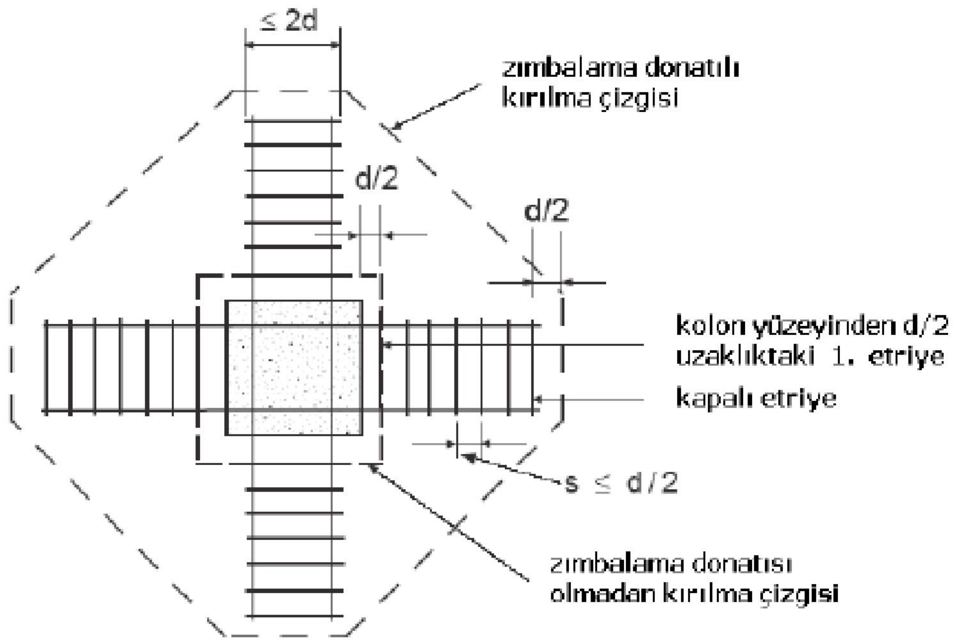
8.3.2 - Punching Shear Reinforcement

The punching shear strength obtained from Equation 8.21 can be increased by using suitable reinforcement or arrangements of steel sections or special steel elements if such increase has been validated by experiments. However for punching shear reinforcement to be effective, the slab thickness must be at least 250 mm. Also the punching shear strength augmented in this manner cannot exceed 1.5 times the value obtained by using Equation 8.21.

in slabs, contribution of rebar to concrete section in limit of 50% enhance. So rebar is used for all column-slab connections to resist of punching failure .

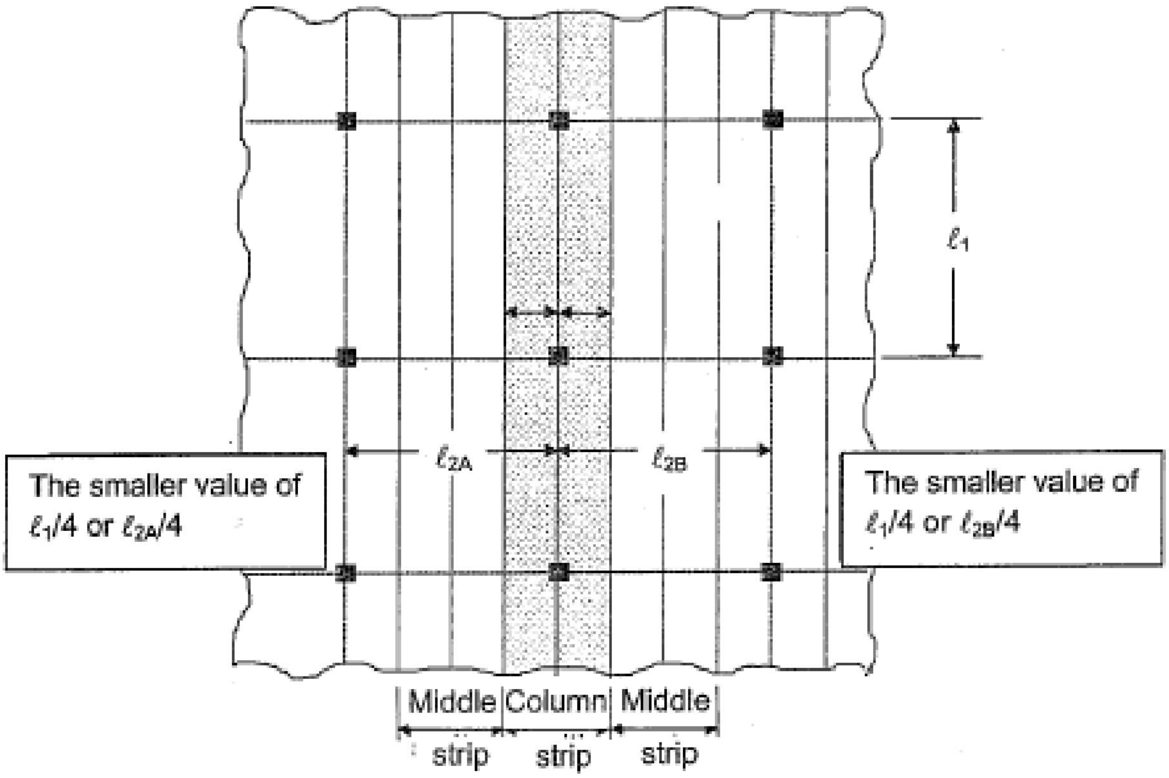
Slab design results, which are solved in Sta4Cad programme, are shown in calculation report appendixes.

Vpr = γ fctd Up d ile belirlenen değerin 1.5 katını aşamaz.



Zımbalama donatısı, kesme kuvvetini etriyelerle kolona taşıdığı için kırılma çizgisini daha geniş kendi kontruna taşımaktadır. Bu yeni kırılma çizgisine göre yeniden zımbalama dayanımı yapılmalıdır. Bu hesaplamada etriyeler dikkate alınmadan yapılmalıdır.

Slab Strips according to TS 500:



a. Slab without beams

Blocks Two way slabs are designed with beams. General slab thickness is chosen 250 mm. Slab thickness is determined as per to TS500 section 11.4.2 equation 11.1. With this equation, slab thickness is calculated 210 mm for 8.40 x 8.40 span.

The depth of a two-way slab with beams cannot be less than the value given by using Equation 11.1.

$$h \geq \frac{\ell_{sn}}{15 + \frac{20}{m}}\left(1 - \frac{\alpha_s}{4}\right) \qquad (11.1)$$

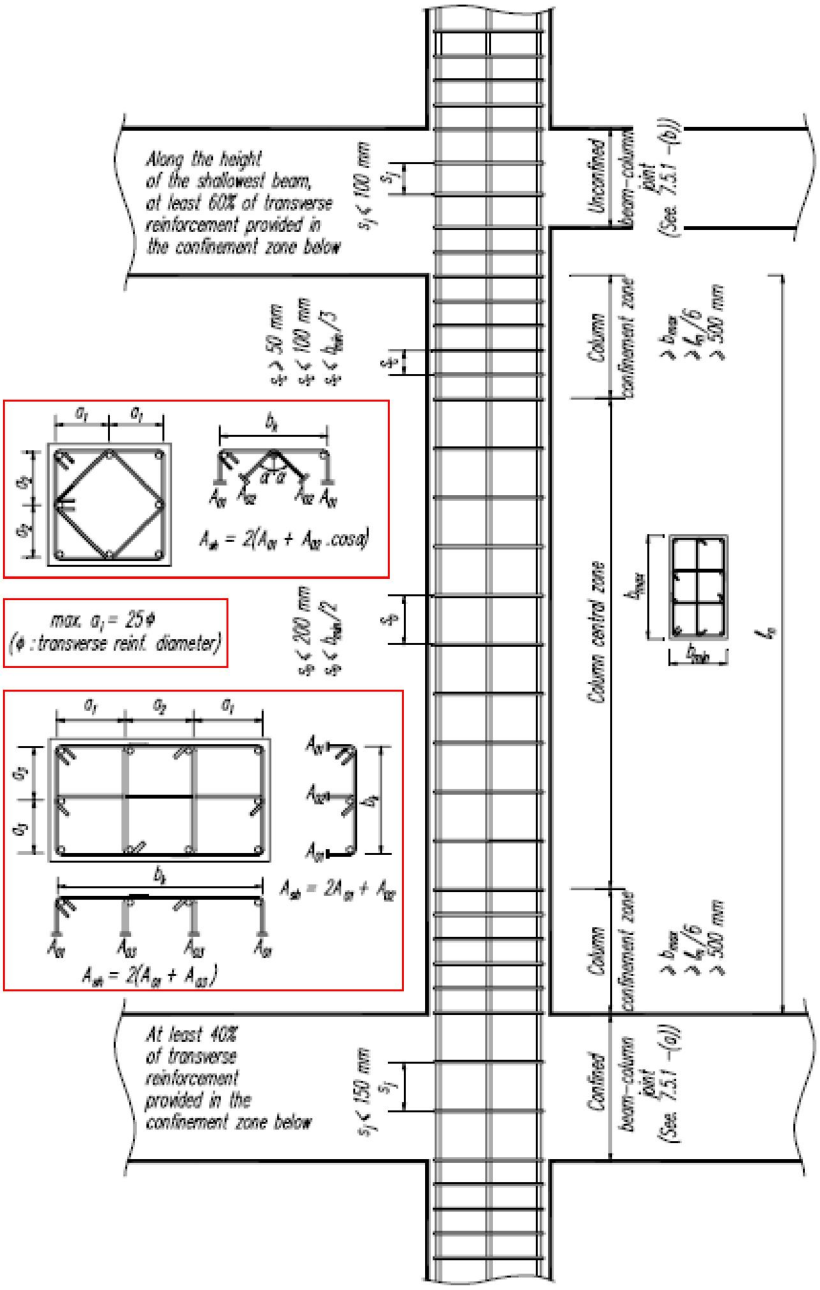
$$h \geq 80mm$$

Due to structural analysis results & economical optimization, slab thickness is chosen 250 mm.

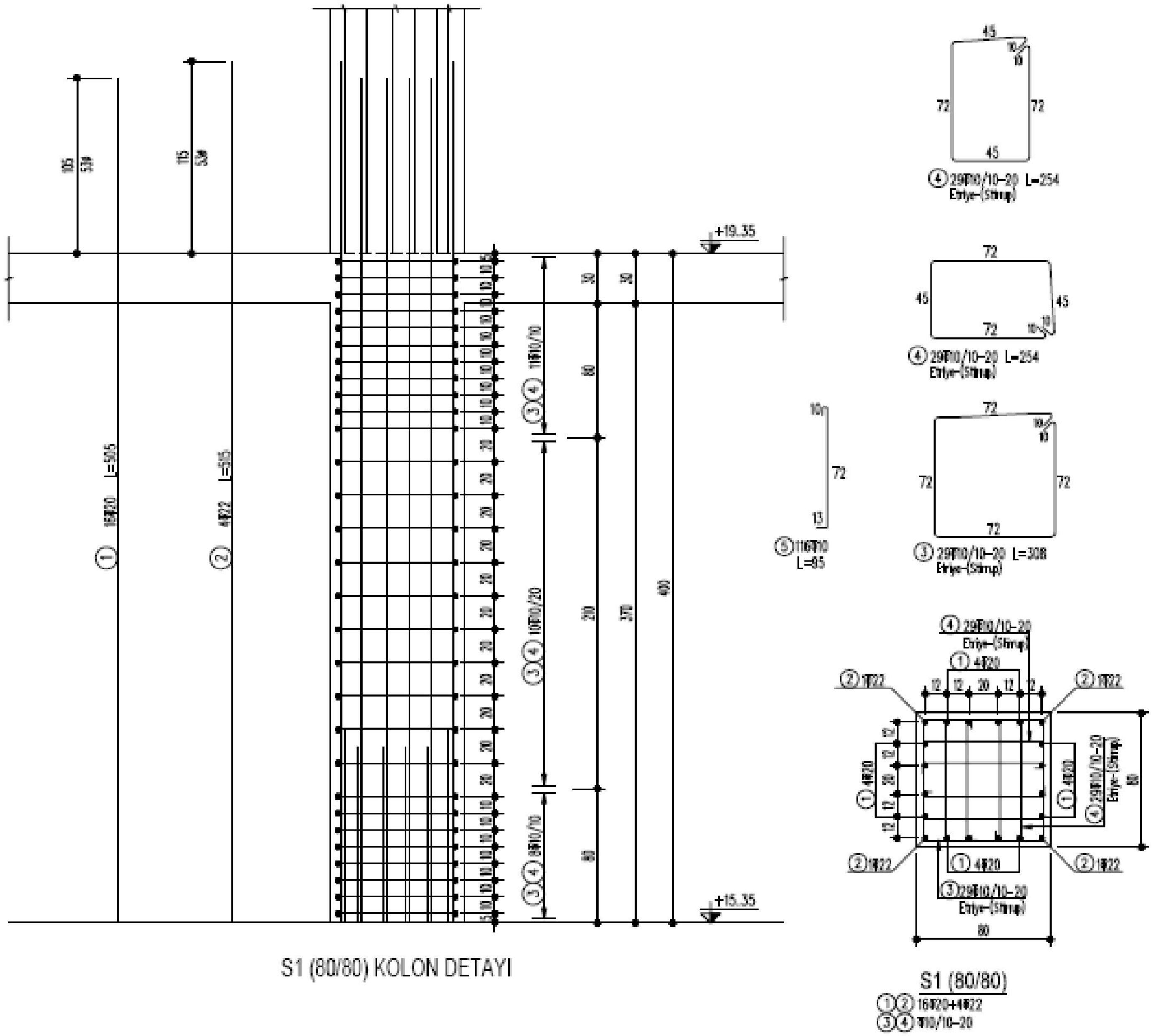
Floor loads are provided in the “Load Analysis” section of this report.

1.2.c. Columns

All columns desingned as per to Turkish EQ Code 2007 Section 3.3 and TS 500 Section 7.4. Minimum and maximum specifications about columns are take into account from these codes. All analysis results are given in the Design Reports section which is performed with STA4Cad.



Column dimensions are given below. All dimensions in cm. 40/70, 40/80, 40/90, 40/100, 40/150, 40/200, 80/80, 90/90, 100/100, 90/100, 90/110, 90/120, 90/130



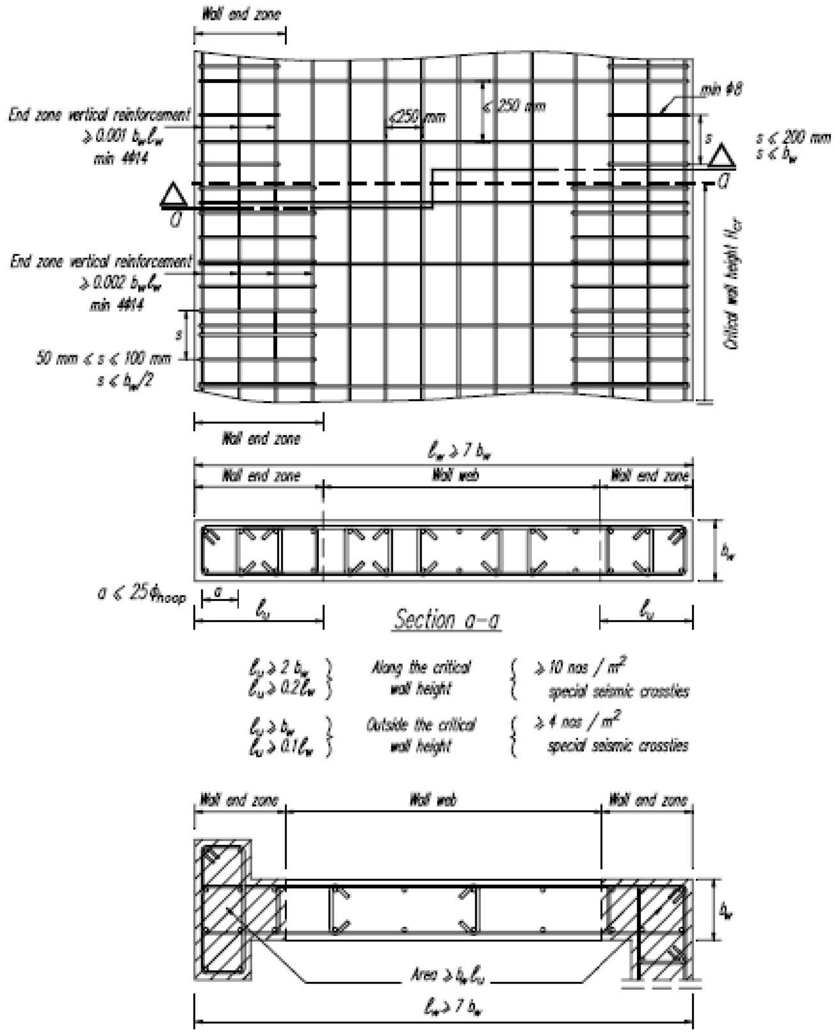
Stirrup spacings, hook details, vertical rebar overlapping lenghts and the other detailed as per to Turkish EQ Code 2007.

1.2.d. ***Shearwalls***

Shearwalls designed as per to Turkish EQ Code 2007, Section 3.6.1. All analysis results given in the Design Reports section which is performed with STA4Cad.

Turkish EQ Code 2007, Section 3.6.3.1 : Total cross section area of each of the vertical and horizontal web reinforcement on both faces of structural wall shall not be less than 0.0025 of the gross section area of the wall web remaining in between the wall end zones.

Thus, all shearwall reinforcements calculated according to this rule,detailed section is given below.



*Minimum vertical reinforcement at the wall end zones is calculated as per to code formula (0.002*bw*lw).*

1.2.e. *Raft Foundation*

Mat foundation system is chosen as advised at geotechnical report. Mat foundation is designed in one piece to prevent differential settlement.Foundation statical calculations are shown in calculation report appendixes.Interaction between the structural system & ground is procured by area (shell) spring constants which is obtained from soil investigation & geotechnic report.

“Soil Subgrade Spring Constant” value is assigned at Sta4Cad programme as shown below.

3 LOADS

All of the loads and load effects are taken into account as required by the related standards and codes that the structural system of the building is going to withstand during its service lifetime.

3.1 SELF WEIGHT OF THE STRUCTURAL MEMBERS AND DEAD LOADS

Self weights of the structural members defined in the analytical model are taken into account both in the analysis and design automatically by the software. Dead loads include the weights of the slab coverings, plastering, plumbing, tiling etc. These loads are calculated with the light of TS498 code and based on the unit weights and the dimensions given in the architectural basic design. These loads applied on the members in the analysis model. Dead loads are applied to the analytical model as uniformly distributed area loads and line loads. Assigned area dead loads to analysis model are tabulated below.

	Thickness (m)	**Unit Weight (t / m^3)**	**Weight (t / m^2)**
Lining + Sloping concrete	0.055	2.2	0.121
Isolation			0.054
Ceiling screed / Suspended ceiling			0.050
		TOTAL	**0.225**

3.1.1 NON LOAD BEARING WALL WEIGHTS

All non load bearing wall weights applied with the uniform surface live loads.

3.2 LIVE LOADS

The live loads of the building areas are determined in accordance with the service conditions. Live loads are applied to the analytical model as uniformly distributed area loads and frame line loads. Area assigned live loads are shown graphically in later sections of this report.
There are many corridors and different technical equipment loads. So the live loads assumed as 500 kg/m^2.

3.3 WIND LOADS

Wind loads calculated according to TS498. All wind speeds and suction forces whic is used for calculation, is shown below. Wind loads and wind load calculations changes with the building height. At the comparison stage of seismic and wind loads, seismic loads bigger than wind loads. Thus, wind loads didin't use for structural memeber design.

Height from gorund (m)	Wind Speed (m/s)	Suction Force; q (t/m^2)
0-8	28	0.05
9-20	36	0.08
21-100	42	0.11
>100	46	0.13

3.4 EARTHQUAKE LOADS

For the earthquake forces calculation, Specifications for Structures to be Built in Disaster Areas 2007 code used with dynamic lateral force procedure is used, because of building loaction, building height and structural system irregularity.

According to dynamic lateral force procedure, sufficient number of vibration modes is used for the analysis of earthquake loads with the 90% effective participating mass of total building. Dynamic lateral earthquake loads compared with the equivalent lateral force procedure loads and dynamic scale factors multiplied with the new values if it is necessary. Respons spectrum graphic shown below.

Earthquake calculation parameters taken from geotechnical / soil investigation report and Turkhish Earthquake Code. Details given below.

Seismic Zone	3	**A_O**	0.2
Soil Class	Z1	**T_a (second)**	0.10
Soil Group	A	**T_b (second)**	0.30
Importancy Factor (I)	1.5		

GAZİANTEP EARTHQUAKE ZONING MAP

Appendix H4

Earthquake Assessment of Gaziantep Integrated Health Campus

Earthquake Assessment of Gaziantep Integrated Health Campus

TDY 2007 (Turkish Earthquake Code 2007) is used for seismic calculation of new & existing buildins in Turkey. This code uses similar acceleration spectra with international codes as UBC, IBC, ASCE exc.

Dynamic response spectrum analysis – Mode Superposition Method is used for calculation of earthquake as per to Turkish EQ Code 2007.

Seismic forces are calculated in TYDY 2007 by using of coefficients shown below :

- Effective Ground Acceleration Coefficient – Seismic Zone

Seismic Zone	A_0
1	0.40
2	0.30
3	0.20
4	0.10

Gaziantep is in 3 rd region – Ao = 0.2 g

- Building Importance Factor

Purpose of Occupancy or Type of Building	*Importance Factor (I)*
<u>1. Buildings required to be utilized after the earthquake and buildings containing hazardous materials</u> a) Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) b) Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
<u>2. Intensively and long-term occupied buildings and buildings preserving valuable goods</u> a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. b) Museums	1.4
<u>3. Intensively but short-term occupied buildings</u> Sport facilities, cinema, theatre and concert halls, etc.	1.2
<u>4. Other buildings</u> Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

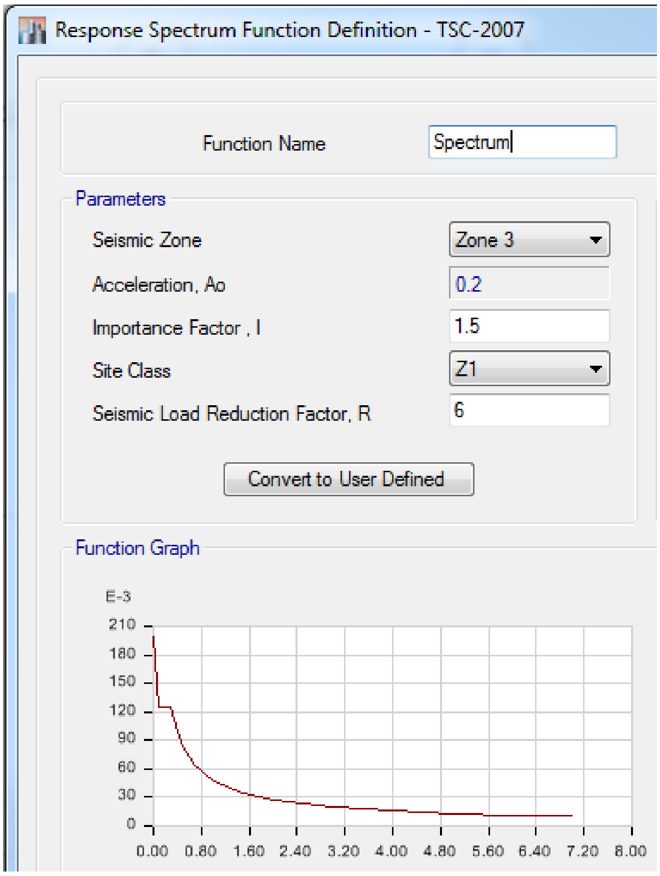
As the Gaziantep Integrated Health Campus is a building to be utilized right after the earthquake, the importance factor that contributes to the design is = 1.5

- Local Site Class – Spectrum Characteristic periods

Local Site Class according to Table 6.2	T_A *(second)*	T_B *(second)*
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

For Gaziantep Integrated Health Campus; Site "Z1" (the site is composed of mainly basalt/rock)

<u>Special Design Spectra for Gaziantep Integrated Health Campus:</u>



<u>Result:</u> Structural system modelling computer programmes as ETABS, SAFE, STA4CAD, which are used in Gaziantep Integrated Health Campus structural project, calculated seismic forces as per to TDY 2007 code. Accordingly, the design is finalized to cover seismic loads calculated in X-Y directions and loads are met with shear walls in both directions in the system. The stirrups agains vertical loads are designed as per the Specification for Buildings
to be Built in Seismic Zones.