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Case Study

Turkey's grand challenge: Disaster-proof building inventory within 20 years☆

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ABSTRACT

Turkey is located in a high seismicity region and has suffered extensive losses due to several major earthquakes that struck its various parts in the past two decades. While earthquakes are associated with damage and loss wherever they may occur, the destructive effects of those in Turkey are exacerbated by the large volume of code incompliant buildings constructed with poor materials and workmanship. As a large scale remedial initiative, Turkey has recently embarked upon a grand challenge of retrofitting or renewing all high-risk buildings within the next 20 years. This multi-million building and multi-billion dollar initiative has inevitably raised activity and debates in diverse disciplines regarding all aspects. This paper focuses on the methodologies and developing technologies for rapid condition assessment and structural evaluation of existing buildings in order to identify and prioritize high-risk buildings and for guiding decisions on retrofitting or renewal.

1. Background and introduction

Protection of people and the built environment from the destructive effects of natural hazards is a worldwide challenge that faces nations at various degrees of significance based on (1) the type of observed hazards; (2) the size of exposure, i.e. the number of people and structures affected by the hazards; (3) vulnerability of the exposure to the impinging hazards. The prediction and mitigation of damage and losses inflicted by natural hazards are among the most active and invested areas of research participated by all nations in proportion with their exposure and resources (Kidokoro et al., 2008).

A quick survey of the most damaging and deadly natural disasters recorded in history reveals that earthquakes occupy a prominent proportion, especially when followed by resulting hazards such as a large fire or a tsunami (Coburn and Spence, 2002). Indeed, earthquakes constitute a primary concern in any country located in a seismic zone. Despite the unceasing attempts to predict earthquakes in more than a hundred years, it is clear that seismic events cannot be predicted accurately enough to issue alarms of imminent damaging earthquakes (Geller, 1997). Nevertheless, significant progress made in fault modeling and probabilistic assessment of seismic hazard combined with performance based structural design and evaluation methods provide the necessary tools for engineers to be prepared for earthquakes (Kramer, 1996; Bozorgnia and Bertero, 2004) although cases of prominent skepticism regarding the validity of probabilistic hazard assessment should be noted (Gulkan, 2013). The meaning of preparedness may naturally differ for different nations based on the size of their economy, their level of investment in infrastructures, and their perception of acceptable risk for the population and

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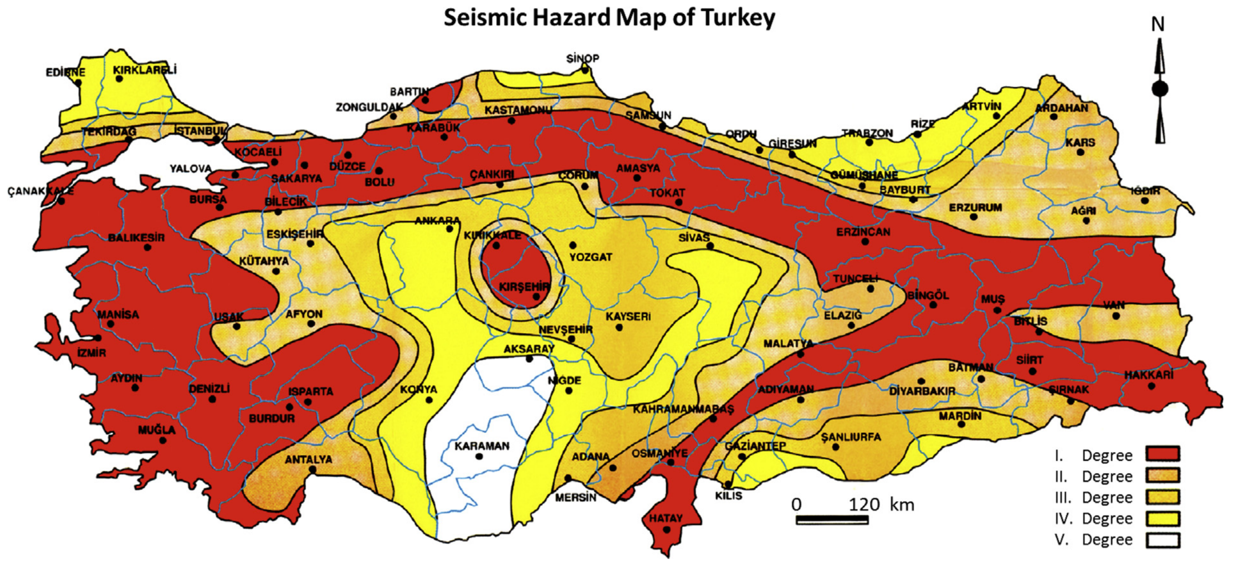


Fig. 1. The seismic hazard map of Turkey.
Source: Republic of Turkey, Prime Ministry Disaster and Emergency Management Presidency, AFAD.

infrastructure (Coburn and Spence, 2002). Although the level of confidence and conservatism used in the aforementioned tools for preparedness may differ in different codes, a generally accepted level of satisfactory performance is (a) for ordinary residential and commercial buildings to survive a design earthquake without collapse; (b) for essential buildings such as schools and hospitals to stay functional after a design earthquake and survive a rare earthquake; and (c) for critical facilities and lifelines to stay fully operational after a design earthquake and stay functional after a rare earthquake.

Turkey is a country known for its unique geography that bridges continents and cultures. It also bridges several tectonic plates including the Eurasian, African, and Arabian plates through the Anatolian plate (Bommer et al., 2002). Interactions between all surrounding plates and the Anatolian plate produce an active seismic region that encompasses most of Turkey as shown in Fig. 1. As a result, earthquakes have been by far the most significant natural hazards in the region. Based on available records, all natural disasters in Turkey since the beginning of the 20th century resulted in 87,000 casualties, 210,000 injuries, and 651,000 heavily damaged or destroyed homes. Earthquakes were responsible for 76% of the damaged or destroyed homes, followed far behind by landslides (10%) and floods (9%) (Ergunay, 2007). Several moderate to devastating earthquakes that occurred in Turkey in the past two decades have claimed nearly 20,000 lives and cost more than $17 billion in direct and indirect losses. The most significant one among these was Kocaeli Earthquake (August 17, 1999, Mw = 7.4) which caused more than 17,000 casualties and cost around $13 billion; and the most recent one was Van Earthquake (October 23, 2011, Mw = 7.2) which caused more than 600 casualties and cost $1–2 billion (Buyukozturk and Gunes, 2002, 2003; Ergunay, 2007; Erdik et al., 2012).

Table 1 shows the distribution of various elements such as land area, population, industrial installations and hydroelectric dams within the seismic zones shown in Fig. 1 (Ergunay, 2007). About half of each element is located in the first degree seismic zone and most of each is located in some degree of seismic zone other than V. Hence, earthquakes affect nearly the entire nation and unless effective mitigation strategies are developed and implemented in a timely fashion, seismic losses will continue to increase in the future.

The concerns highlighted in Fig. 1 and Table 1 are exacerbated by the fact that a very large portion of Turkey's building stock does not comply with either the structural/seismic codes that were effective at the time of their construction, or the ever more stringent modern seismic code enforced today. As a matter of fact, it is often reiterated in the daily press and accepted by the government officials that half to three quarters of existing buildings in Turkey lack the design documents and permits required for their construction. Referred to as illegal construction, these buildings are generally constructed with poor materials and workmanship due to insufficient or no supervision or inspections during construction.

An even more concerning development right after the devastating Kocaeli Earthquake in 1999 was the findings of a scientific study which indicated that the probability of occurrence of another severe and destructive earthquake along the

Table 1
Distribution of various elements within different seismic zones (Ergunay, 2007).

Seismic zone (Fig. 1)		Area (%)	Population (%)	Industry (%)	Dams (%)
I	(pga = 0.4 g)	42	45	51	46
II	(pga = 0.3 g)	24	26	25	23
III	(pga = 0.2 g)	18	14	11	14
IV	(pga = 0.1 g)	12	13	11	11
V	(pga < 0.1 g)	4	2	2	6

North Anatolian Fault, near Istanbul, was 62 ± 15% within the next 30 years (Parsons et al., 2000). Now almost 15 years after this study, the progress made toward preparing for the next big earthquake has not gone much beyond pilot studies of regional risk assessment and seismic retrofitting of a limited number of essential buildings. Istanbul and the adjacent areas constitute the most densely populated and most industrialized region in Turkey, hence, the heart of Turkish economy. It is impossible to make reliable estimates of seismic losses due to aforementioned complexities but studies based on scenario earthquakes close to Istanbul roughly estimate 30,000–40,000 heavily damaged buildings – corresponding to 5% of the building stock in the region – with 5000–6000 of them collapsed, 30,000–50,000 casualties and $11 billion in direct losses due to damage to buildings (Erdik et al., 2003; Strasser et al., 2008).

2. The grand challenge and the scope

The grim picture portrayed by the facts laid out in the preceding section has long called for immediate action in terms of putting greater emphasis and investing more resources on increasing disaster resilience of the building infrastructure in Turkey. During the early years after Kocaeli Earthquake in 1999, the main priority was the recovery of the affected region by means of building housing for those in need of shelter and restoring the interrupted economic activity. In addition to disaster relief funds, the recovery efforts were funded by international aid and special temporary taxes which later became permanent. In the following years, the above-mentioned call was largely subdued by concerns related to the national economy. Seismic mitigation efforts attracted relatively modest attention and budget for more than ten years during which related research studies were skewed more toward seismic risk assessment than development and implementation of effective retrofit/renewal strategies. Isolated cases of larger scale initiatives such as the Istanbul Seismic Risk Mitigation and Emergency Preparedness Project (ISMEP) must be duly noted (www.ipkb.gov.tr). This ongoing project was initiated in 2005 with a €310 million budget which rose to €1 billion 213 million by 2012, all funded by loans from international funding institutions. Expected to be completed in 2018, main components of the project include (a) enhancing emergency preparedness; (b) seismic risk mitigation for priority public facilities; (c) enforcement of building code. Although a large and comprehensive project, the limited retrofit/renewal content of ISMEP is far from addressing the overall need for seismic mitigation.

Van Earthquake of 2011 turned out to be a major wake-up call for the officials to finally take action on the seismic deficiency of the building stock. The launch of a massive initiative was announced right after the earthquake and six months later Law No 6306 "Transformation of areas under disaster risk," often referred to as the Urban Transformation Law, was approved by the Parliament to be implemented by the Ministry of Environment and Urbanization. The law sets the ground rules and procedures regarding the identification and renewal of high risk buildings as well as high risk areas, the latter of which is a controversial authority given to the cabinet. To facilitate rapid voluntary implementation, the law does not seek consent of all occupants in a building identified as high risk, but rather seeks agreement of two thirds of the occupants sufficient to implement the renewal process. Objections are handled by designated local technical commissions formed by ministry employees and university professors, whose decision is final. Once a building is deemed high risk, the administration has the authority to require its evacuation and demolition even without the consent of the occupants, or to have it demolished for reconstruction. Rent assistance and several fee deductions or waivers as well as financing options are offered to the occupants during the renewal process.

It is estimated that about one third of the nearly 20 million occupancy units in Turkey has insufficient seismic resistance and need retrofitting or renewal. This estimation is in agreement with, or perhaps inspired by, results of regional loss estimation studies (Ansal et al., 2009). The cost of urban transformation is roughly estimated as $500 billion and the time to completion is ambitiously set as 20 years. These figures make this initiative, if adamantly pursued, one of the largest reconstruction projects in history and Turkey one of the largest construction markets for the next two decades.

Besides the natural excitement in the construction industry and the real estate business, the urban transformation initiative has stirred heated discussions and debates in diverse disciplines regarding its all facets. The pros and cons of the urban transformation concept in general and the initiative in particular are scrutinized from sociological, political, legal, environmental, urban planning and human rights perspectives (Kuyucu and Unsal, 2010; Ozus et al., 2011; Balaban, 2012; Uysal, 2012; Colak, 2013; Karaman, 2013; Elicin, 2014).

The scope of this paper is limited to the civil engineering aspect of the urban transformation initiative. Screening millions of structures to identify those with insufficient seismic resistance requires rapid, reliable and economical tools for condition assessment and structural evaluation. Statistics and important characteristics of the existing building stock in Turkey are presented first, followed by the codes and procedures currently in use. Methodologies and brewing technologies for rapid and reliable condition assessment and structural evaluation are discussed for their potential use in the identification and prioritization of high-risk buildings as well as for providing decision support regarding their retrofitting or renewal.

3. Characteristics of the building stock in Turkey

3.1. Population and structural characteristics

Assessment of the size and characteristics of the exposure subjected to the seismic hazard shown in Fig. 1 is an important component of seismic risk assessment and mitigation. In this section, brief information is provided about the numbers and important material and structural characteristics of existing buildings in Turkey.

The last comprehensive building census in Turkey was performed in 2000 by the State Institute of Statistics (TSE, 2000), now known as the Turkish Statistical Institute (TUIK). Although a more recent 'Population and Housing Census' was performed in 2011, this study involved a sample of the population, albeit a large one, and not the whole population (TUIK, 2011) with the objective to produce the population and housing statistics requested by the United Nations that could not be derived from the 'Address based Population Registration System' (APRS) established in 2007. While the 2011 census does not provide accurate information on population and number of buildings, the former can be obtained from APRS and the latter from numbers of building occupancy permits issued since 2000, which obviously exclude those constructed without permits. Fig. 2 shows the total number of buildings, occupancy units, and the population in Turkey compiled from population and building census results, the occupancy permit data and the APRS data obtained from TUIK (www.tuik.gov.tr). As of 2010, the average size of households is 3.9 and the average number of occupants in a building is 8.7, where the average number of units per building is 2.25.

Distribution of the occupancy units by building age is shown in Fig. 3 for Istanbul and Turkey based on the 2011 Building and Housing Census report (TUIK, 2011). The information in the figure combined with the knowledge of seismic code development provides helpful information regarding the expected performance of buildings based on their construction era.

The first national seismic design code in Turkey was published in 1944 which was revised seven times at various dates until the most recent version published in 2007. Early versions of the code included mostly prescriptive rules. The 1975 version was the first to closely resemble modern seismic codes, whereas the 1998 version can actually be considered as one. The currently enforced 2007 version is the first to include state of the art performance based evaluation concepts and it is currently under revision for further improvements.

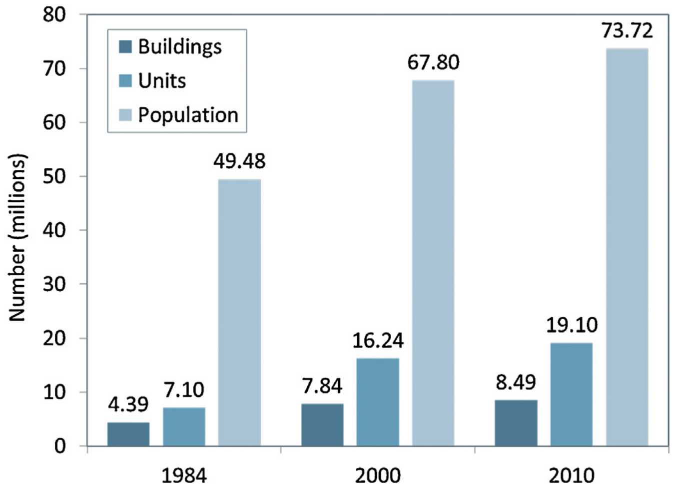


Fig. 2. Total number of buildings and occupancy units in Turkey.
Source: Turkish Statistical Institute, TUIK.

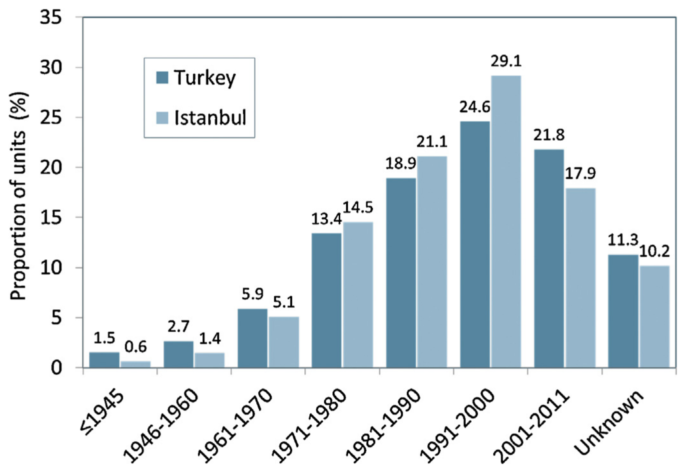


Fig. 3. Proportions of occupancy units by the construction year of buildings in Istanbul and Turkey.
Source: TUIK.

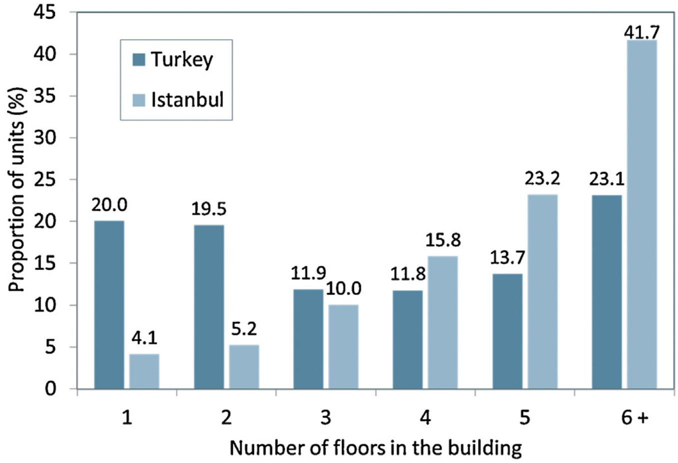


Fig. 4. Distribution of occupancy units by the number of floors in the building in Istanbul and Turkey.
Source: TUIK.

Possibly influenced by the above knowledge of seismic codes, performance grouping of buildings with respect to age is generally performed as pre- and post-1980 (Erdik et al., 2003; Strasser et al., 2008; Ansal et al., 2010). The latter group can further be divided as pre- and post-2001 considering that Law No 4708 Construction Inspection Law passed in 2001 has led to better quality control of buildings constructed after this date. According to this grouping, 23.4% of the existing occupancy units were constructed in or before 1980, 43.5% in 1981–2000, and 21.8% in or after 2001, while the construction year of 13.1% is unknown (TUIK, 2011).

Height of buildings is an important characteristic in that combined with the local soil conditions it may have a strong influence on the level of seismic base shear and hence the level of damage and loss. The distribution of all occupancy units in Istanbul and Turkey by the number of floors in the building is shown in Fig. 4 (TUIK, 2011). Due to high population density and scarcity of land for construction, more than 40% of the occupancy units in Istanbul are in buildings six floors or higher. The average number of floors in Turkey was determined as 4.0 while that in Istanbul is much higher with an average value of 5.7.

The type of material and structural system of existing buildings are important indicators of their vulnerability under the effects of seismic actions. Vulnerability measures, whether empirical or based on rigorous analyses, are often expressed in conjunction with buildings' materials and structural system type, their age group, and height group (Erdik et al., 2003; Strasser et al., 2008; Ansal et al., 2010). Fig. 5 shows the distribution of buildings in Istanbul and Turkey by their structural type. Data for 1984 and 2000 were obtained from the 2000 Building Census report (TSE, 2000) while those for 2010 was

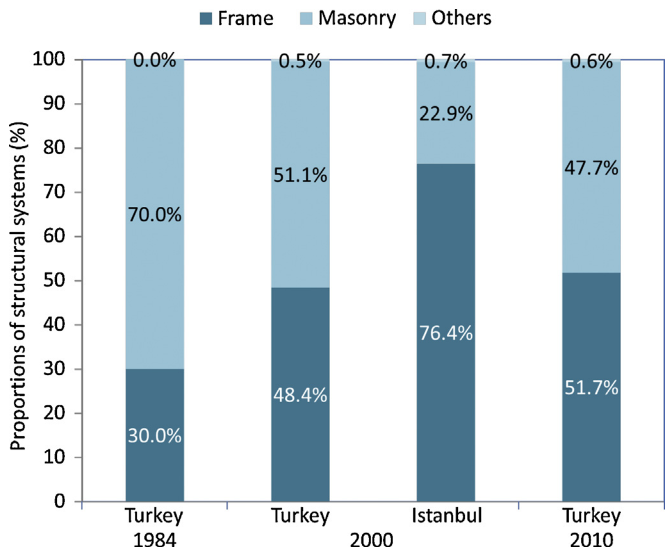


Fig. 5. Distribution of buildings in Istanbul and Turkey by their structural system.
Source: TUIK.

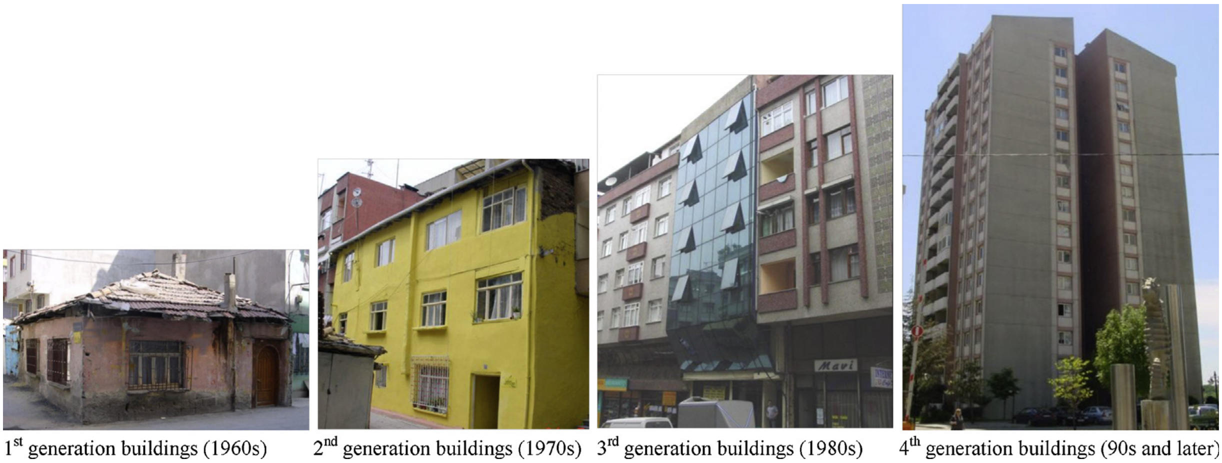


Fig. 6. Representative buildings by their construction era in Istanbul's Zeytinburnu region.
Source: Republic of Turkey Ministry of Environment and Urbanization, General Directorate of Infrastructure and Urban Transformation Services, CSB-AKDHGM.

compiled from building occupancy permits issued between 2000 and 2010, which should be considered as approximate values due to minor discrepancies in data obtained from different tables. Frame type structures, predominantly reinforced concrete frame buildings accounted for more than 90% of buildings constructed in 2000–2010. Hence, the proportion of frame structures in the building inventory is continuously increasing as can be seen from Fig. 5. As of 2010, more than half the structures in Turkey were of frame type compared to less than one third in 1984. This ratio is much higher in Istanbul where more than three quarters of all buildings have frame type structural systems.

The information provided in Figs. 3–5 about the age, height, and structural systems of existing buildings does not allow generalizations based on these characteristics. Nevertheless, Fig. 6 shows sample photos of frequently encountered buildings constructed in different eras in Istanbul's Zeytinburnu district to give the reader a rough idea about the building profile in densely populated urban areas in Turkey.

3.2. Materials characteristics

Many factors play a role in the large scale destruction experienced during recent major earthquakes in Turkey. Among these, poor quality of construction materials is almost invariably cited as an important contributing factor in reports of heavily damaged or collapsed buildings. The current seismic code specifies a minimum characteristic compressive strength of 20 MPa for concrete used in buildings in seismic zones. However, the concrete strength in a majority of the existing buildings constructed before 2000, where concrete was mixed on site, falls below this minimum requirement, and values below 10 MPa are not uncommon. Fig. 7 shows the characteristic compressive strength distribution of concrete samples obtained from a large number of reinforced concrete buildings in Istanbul and surrounding cities constructed before 2000 mostly using site-mixed concrete (Bal et al., 2008). The results have shown that the characteristic compressive strength of concrete in more than two thirds of the buildings was less than the required minimum (20 MPa). The percentage of buildings with a characteristic concrete strength 8 MPa or less was 21.3% which corresponded to about 135,000 buildings

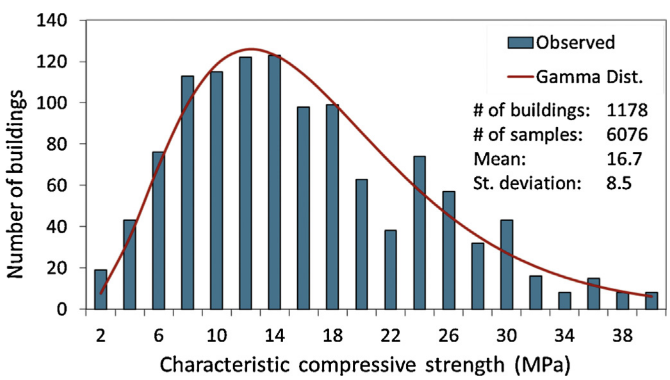


Fig. 7. Distribution of concrete strength in buildings constructed before 2000 mostly with site-mixed concrete in Istanbul and surrounding cities.
Source: Reproduced from Bal et al. (2008).

considering the building population in the studied region. Perhaps even more dramatically, the percentage of buildings with a characteristic concrete strength 4 MPa or less was 5.3%, corresponding to nearly 34,000 buildings in the region, which provides a clear indication of the size of destruction that can be caused by a major earthquake in the region.

Use of ready-mixed concrete in construction and improved site supervision enforced by the Construction Inspection Law (Law No 4708) passed in 2001 have improved the quality of concrete in buildings constructed after 2000. Bal et al. (2008) reported that 28-day strength of concrete samples obtained during construction of 433 buildings showed a log-normal distribution with a mean concrete strength of 24.9 MPa and a standard deviation of 2.1 MPa.

As important as the strength of concrete is the strength and ductility of the reinforcing steel used in buildings. Quality of steel reinforcement receives less attention than that of concrete since it is not produced on site and a relatively higher level of quality control is typically enforced during its production. The current seismic code requires use of deformed steel reinforcement in structures with the exception of shear reinforcement and slab reinforcement where smooth bars can be used. Typical types of deformed and smooth steel bars are S420 and S220, respectively, designations of which indicate their required characteristic yield strength in MPa. Reinforced concrete buildings constructed before the early 1970s involved exclusive use of S220 type steel as the use of S420 type steel started with the first publication of the related standard TS 708 (2010) in 1973. Between the 1970s and late 1990s, use of S420 type steel increased almost linearly. In buildings constructed after 2000, S420 type steel was used almost exclusively as most major steel manufacturers terminated the production of S220 type steel in late 1990s (Bal et al., 2008).

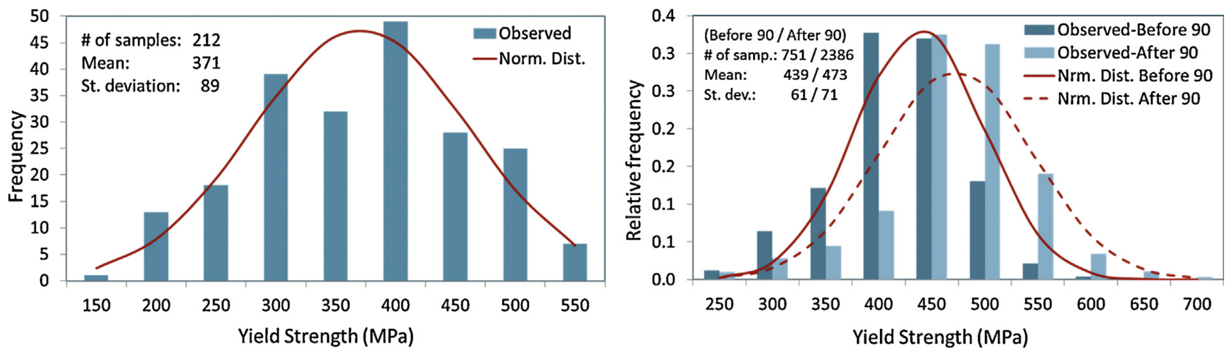
Yield strength distributions of S220 and S420 type steel samples obtained from existing buildings are shown in Fig. 8. These distributions, obtained from fairly large sets of data, were compiled by Bal et al. (2008) from data reported in the literature. The distribution of S420 type steel was presented as pre- and post-1990 due to the knowledge of significant difference in the characteristics of S420 type steel produced before and after 1990.

It should be emphasized that the data presented in Fig. 8 shows the distribution of yield strength and not the characteristic yield strength. The definition of the characteristic yield strength in TS 708 was adopted from EN 10080 as "... the lower or upper limit of the statistical tolerance interval at which there is a 90% probability ($1 - \alpha = 0.90$) that 95% ($p = 0.95$) or 90% ($p = 0.90$) of the values are at or above this lower limit, or are at or below this upper limit, respectively." This definition refers to 90% confidence interval and for a large number of samples ($n > 1000$) corresponds to the 5%-fractile value to be compared with the required minimum. Based on the sample size and yield strength distribution of S220 type steels in Fig. 8a, the characteristic yield strength was calculated as 212 MPa, which is slightly below the code requirement. Similar calculations for the S420 type steels in Fig. 8b produced characteristic yield strength values of 334 and 357 MPa for pre- and post-1990 production, respectively, both of which are significantly below the 420 MPa minimum required by the code.

3.2.1. *Just how low is too low?*

Quantitative assessments of concrete and steel characteristic strengths presented in the preceding section point to numerous cases of significant deficiencies compared to respective code specified minimums. These deficiencies constitute a major contributing factor to the extent of destruction and losses caused by recent major earthquakes in Turkey. The lower end region of the concrete characteristic strength distribution shown in Fig. 7 especially, which obviously excludes even more extreme cases of low quality concrete shown in Fig. 9, sends out serious warning signs of the devastation that may still lie ahead in case of a major earthquake near Istanbul.

A crude estimation of the proportion of substandard buildings in Turkey based solely on the material characteristics shown in Figs. 7 and 8 would yield a figure well above 50% just by comparing materials characteristic strengths with code required minimums. The real challenge, however, is to estimate the proportion that is likely to suffer heavy damage or collapse due to a design earthquake. This assessment requires seismic performance evaluation and cannot be based on materials information only. Still, one cannot help but ask: "Just how low is too low when it comes to materials strength?" Is there a minimum strength below which is unacceptable even when all else conforms to the codes and specifications? From a



(a) S220 type smooth bars (b) S420 type deformed bars produced before and after 1990

Fig. 8. Yield strength distributions of S220 and S420 type reinforcing steels used in existing buildings.
Source: Reproduced from Bal et al. (2008).

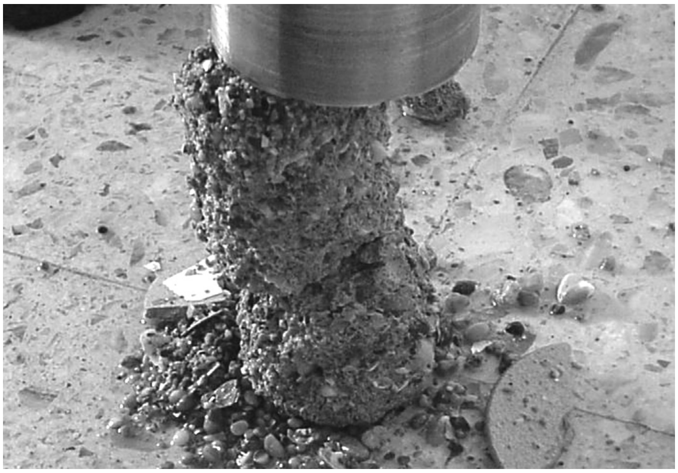


Fig. 9. An extreme case of low quality concrete – a sample crumbled during coring.

mechanics perspective, is there a minimum concrete strength below which the reinforced concrete theory does not apply due to violation of underlying assumptions such as perfect bond or ultimate concrete strain? Finally, how do durability problems such as aging of concrete and corrosion of steel reinforcement be factored in the assessment of minimum acceptable strength?

The above-posed questions do not have easy answers and the author is unaware of targeted studies in this context. It is obvious that the quality of the sample shown in Fig. 9, which was taken from an existing two-story industrial building, is unacceptable, but this sample is not even represented in the distribution in Fig. 7. It is conceivable that a region in the lower portion of the distribution in Fig. 7 could be deemed unacceptable without any considerations beyond concrete strength, but the limit of this region should be selected rationally with proper justification.

Certain real-life occurrences and research results can perhaps be analyzed to search for guidance regarding the above presented discussion and posed questions. As one would fear based on the presented material strength distributions, there were cases of buildings that collapsed under their own weight in Turkey, caused by a combination of factors including low quality of materials. Two recent cases were the collapse of an 11-story reinforced concrete building in city of Konya in 2004 and that of a 5-story building in Istanbul in 2007. The buildings were constructed in 1997 and early 1980s, respectively. Materials investigations have shown that the average concrete strength in both buildings was approximately 9 MPa, with test results as low as 6 MPa, which can be conservatively taken as the characteristic strength. It should be noted that nearly 12% of the buildings included in Fig. 7 had a concrete characteristic strength of 6 MPa or less.

A more representative guide would be the damage statistics from recent major earthquakes. Post-earthquake surveys after the 1999 Kocaeli Earthquake have revealed that approximately 12% of the occupancy units in the strongly affected regions (Kocaeli, Sakarya, Yalova and Bolu) were heavily damaged or collapsed (Gunes et al., 2006). This simply corresponds to a characteristic strength of 6 MPa or less in the distribution shown in Fig. 7.

An interesting study recently completed at Istanbul Technical University (ITU) shed some light on the performance of reinforced concrete column elements with very low strength concrete (4 MPa), corroded (up to 28% section loss) smooth steel reinforcement (S220), and insufficient development length – a worst case scenario which is far too commonly encountered in Turkey (Goksu, 2012; Inci et al., 2013). Fig. 10a shows a column specimen subjected to accelerated corrosion and a corroded sample of steel reinforcement obtained from the column specimen is shown in Fig. 10b. The backbone curves obtained from cyclic flexural testing of column specimens with corrosion up to 12% section loss are shown in Fig. 10c. In the figure, transverse flexural load is normalized with respect to the calculated flexural capacity of the column after corrosion. Hence, the section loss due to corrosion is taken into account in the calculations. The objective here is not to assess the reduction in the column capacity due to corrosion, but to investigate the deviation from the calculated behavior due to the combined effects of corrosion, low concrete strength, and insufficient development length. When the backbone curves shown in Fig. 10c are compared from this perspective, it can easily be discerned that the combined effects of the said negative features reduce both the load capacity and ductility of the column specimen. Looking on the bright side, however, one could nevertheless consider it a somewhat pleasant surprise that column specimens with such serious deficiencies can still produce backbone curves that display the general characteristics of a healthy one at a reduced scale with nearly half the calculated flexural capacity and more importantly some limited degree of ductility – an essential and potentially life-saving feature from a seismic performance viewpoint.

A study of the cases presented above in three groups lead to some conflicting conclusions. In the first group, buildings made of concrete having 6 MPa characteristic strength collapsing under their own weight could suggest that a characteristic concrete strength value higher than 6 MPa should be the minimum acceptable limit with no further considerations. In the second group of cases, the proportion of heavily damaged or collapsed buildings in the hardest hit regions by a major

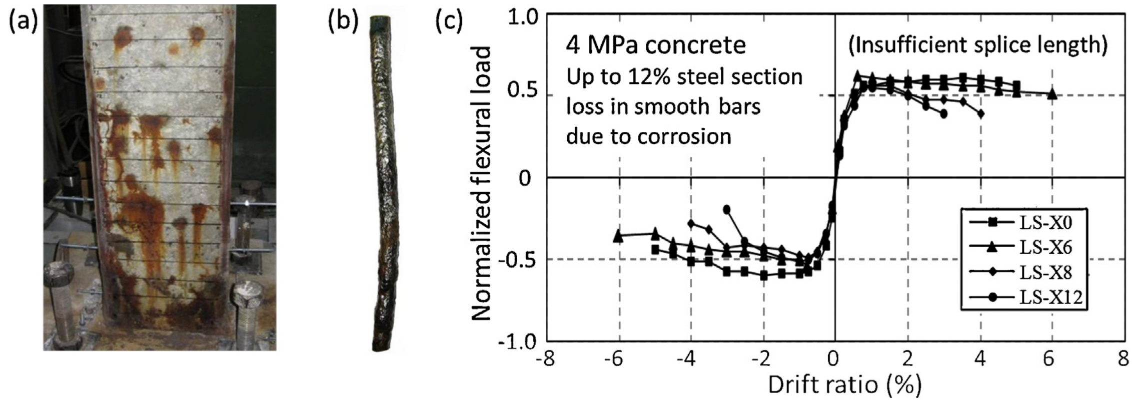


Fig. 10. Reinforced concrete column subjected to accelerated corrosion (a), samples of corroded smooth (S220) and deformed (S420) steel reinforcement (b), and normalized backbone curves obtained from cyclic flexural testing of column specimens (c).
Source: Goksu (2012) and Inci et al. (2013).

earthquake was equivalent to a concrete characteristic strength of 6 MPa or less in a distribution obtained from the same region. Despite the shared value of characteristic strength, the type and severity of loading actions are dramatically different in these two groups of cases. A third group showed that columns made of concrete with a strength as low as 4 MPa reinforced with smooth steel bars with insufficient development length corroded to a section loss of 12% can still display certain behavioral characteristics that should not be discounted form seismic performance viewpoint.

The question posed in the heading of this section, after a not so brief discussion, essentially remains unanswered. There is no doubt that concrete characteristic strength values as low as 2 MPa in existing buildings as shown in Fig. 7 seem frighteningly low and utterly unacceptable. The presented cases and discussions demonstrate, however, that it would not be properly justified to make judgment calls about safety of existing buildings without knowledge of their structural characteristics such as the number of floors, size of members, amount and detailing of reinforcement, etc. Vice versa is also true since safety evaluation based only on structural characteristics without knowledge of material quality and essential properties would be incomplete and potentially wrong.

4. Condition assessment and evaluation of existing buildings

The preceding sections have established the nature and size of the problem and the essential components of a proper methodology. Turkey's grand challenge involves seismic safety evaluation of nearly 9 million buildings with 20 million occupancy units (Fig. 2) and retrofit or renewal of those with insufficient seismic resistance within 20 years. The proportion of reinforced concrete frame buildings, the highest risk group and hence the focus of this paper, is more than half in Turkey and more than three quarters in Istanbul (Fig. 5). More than two thirds of the older buildings are made of concrete with characteristic strength below 20 MPa, the code required minimum (Fig. 7). The characteristic yield strength of reinforcing steel is also below the respective code required minimums to various extents (Fig. 8).

Official estimates indicate that approximately one third of the nearly 20 million occupancy units have insufficient seismic resistance and need retrofitting or renewal. This estimation is in agreement with that reported by regional studies based on scenario earthquakes (Ansal et al., 2009). Actual damage observed during recent major earthquakes, however, was generally well below the estimated level except for very few near fault regions where most unfavorable effects were combined (Spence et al., 2003; Bird et al., 2004). The proportion of heavily damaged or collapsed occupancy units due to the Kocaeli Earthquake was 12% in the strongly affected regions, about 5% in the wider region subjected to moderate-to-strong ground shaking, and about 2% in all affected regions (Gunes et al., 2006). When these statistics are compared to the official estimate, the difference is equivalent to millions of buildings and billions of dollars in retrofit and renewal costs. Hence, it is vitally important that this massive undertaking incorporates reliable methodologies of condition assessment and safety evaluation so that the buildings with insufficient seismic resistance can accurately be identified.

The state of the art international documents outlining the procedures of condition assessment and structural evaluation include ASCE/SEI 41-13 (2014) and Eurocode 8-3 (EN 1998-3, 2005) effective in North America and Europe, respectively. While the latter is also in effect in Turkey as an adapted standard, provisions in Chapter 7 of ABYBHY (2007), hereafter referred to as the Turkish Seismic Code (TSC), is almost exclusively used for the evaluation and strengthening of existing buildings in Turkey. Although not as comprehensive and scrutinized as its international counterparts, this chapter includes the essential components of modern seismic safety evaluation procedures including performance based evaluation.

The provisions of TSC Ch. 7 on condition assessment of existing reinforced concrete buildings are summarized in Table 2. Similar to the provisions of ASCE/SEI 41 and Eurocode 8-3, the assessment is based on three different knowledge levels as limited, intermediate and comprehensive. When calculating element capacities, the in situ concrete and steel strength values defined in Table 2 are used without factoring with any coefficients specified in respective codes, but instead a

Table 2
Provisions for condition assessment of existing reinforced concrete buildings in TSC Ch. 7.

Knowledge level	Geometry	Member details	Materials
Limited	Structural system plan views produced through field work. Must be detailed enough to build a computational model and must include structural irregularities/weaknesses and interaction with neighboring structures. Foundation system identified through test pits	Structural drawings do not exist. Reinforcement details assumed to comply with the required minimums at the date of construction. Spot checks for verification by exposing reinforcement in 10% of columns and shear walls and 5% of beams, at least one on each floor. Nondestructive inspection of reinforcement layout in 20% of the remaining members using covermeter. Ratio of existing reinforcement to minimum required expressed as 'reinforcement realization factor' separately for beams and columns and assumed to apply to the rest of members	At least 2 concrete core samples per floor from columns or shear walls, the lowest result obtained from compression tests used as 'in situ concrete strength'. Reinforcing steel type determined from spot checks and 'in situ reinforcement yield strength' assumed equal to corresponding code required minimum characteristic strength. Corrosion observed in reinforcement during spot checks marked on the plan drawings and considered in the analyses
Intermediate	If technical drawings available, checks made for verification. Otherwise, above provisions apply. Gathered information must allow accurate calculation of building mass	If structural drawings not available, spot checks performed on 20% of columns and 10% of beams. Otherwise above provisions apply. Reinforcement realization factors calculated only in case of mismatch between building and structural drawings, and used with a maximum value of 1 in member capacity calculations	One concrete core sample per 400 m^2 area, minimum 3 per floor and 9 per building. Concrete strength determined as 'mean–std. deviation'. Distribution of concrete strength can be determined through core correlated rebound hammer tests or similar NDT methods. Reinforcement provisions same as above
Comprehensive	Structural drawings exist. Verifications checks made and if significant deviations detected, drawings disregarded and intermediate knowledge level provisions apply	Structural drawings exist. Above provisions apply for verification checks	Provisions for concrete same as above except for one core sample per 200 m^2 area. For each reinforcement type identified during spot checks, a sample tested for conformance to project specifications. If conforms, above provisions apply. Otherwise, at least three more samples of each type tested and the lowest result used as yield strength

knowledge level factor is applied to the calculated member capacity. This factor is specified for limited, intermediate, and comprehensive knowledge levels as 0.75, 0.90, and 1.00, respectively. Hence, for structures identified to have insufficient seismic resistance, increasing the knowledge level through additional investigations may lead to a more favorable evaluation outcome due to reduced uncertainty in capacity calculations.

The inspection and sampling procedures mentioned in Table 2 are demonstrated in Fig. 11 ordered from nondestructive to destructive methods. Use of nondestructive testing (NDT) methods in condition assessment of buildings is encouraged in all major codes but the capabilities and reliability of these methods regarding in situ strength assessment of construction materials are not yet sufficient for them to replace destructive methods (Maierhofer et al., 2010; Buyukozturk et al., 2011). For this reason, use of NDT methods is generally allowed in conjunction with destructive methods for in situ strength assessment so that a reliable correlation can be obtained. TS EN 13791 (2010) standard provides the procedures for use of various NDT techniques for in situ strength assessment of concrete in conjunction with concrete core sampling as a destructive but relatively more reliable method.

The condition assessment studies summarized in Table 2 require that the gathered structural and materials information must be detailed enough to allow computational modeling of the building. Once this is achieved, structural evaluation of the building can be performed using either linear or nonlinear methods described in the TSC. Although the principles of performance based seismic evaluation are based on combined and interactive consideration of seismic demand and the structural capacity obtained from nonlinear analyses, the code allows for seismic performance evaluation of buildings based on linear methods. This is done by the help of member capacity ratios, obtained by dividing member forces calculated from equivalent lateral base shear using linear methods, by the member capacities calculated using in situ materials strength values (see Table 2).

Seismic performance evaluation based on nonlinear methods constitutes a more proper approach to performance based evaluation since the stress redistributions among members are taken into account in the analysis. A conceptual methodology is illustrated in Fig. 12. The capacity curve obtained from nonlinear static (pushover) analysis of the building is shown in Fig. 12a together with the approximate damage states associated with different points on the curve. A convenient means for

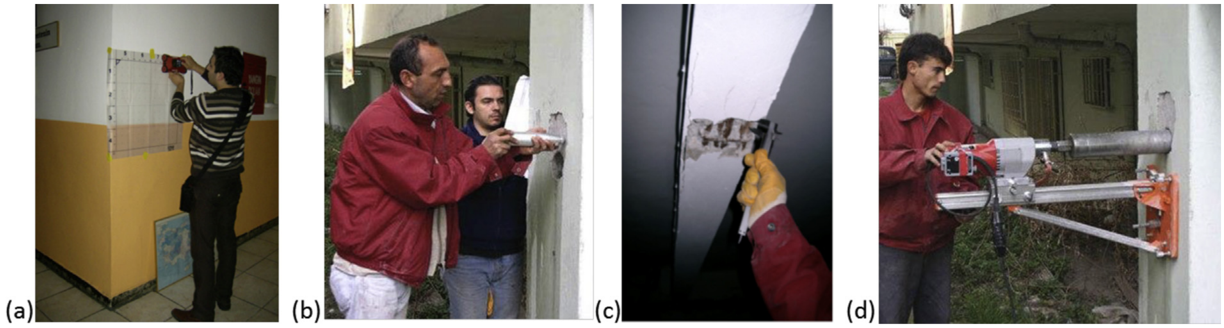


Fig. 11. Methods commonly used for condition assessment of existing buildings ordered from nondestructive to destructive: (a) covermeter, (b) rebound (Schmidt) hammer, (c) spot check of reinforcement number, size and splice length, (d) concrete core sampling.
Source: Republic of Turkey Ministry of Environment and Urbanization, General Directorate of Infrastructure and Urban Transformation Services, CSB-AKDHGM.

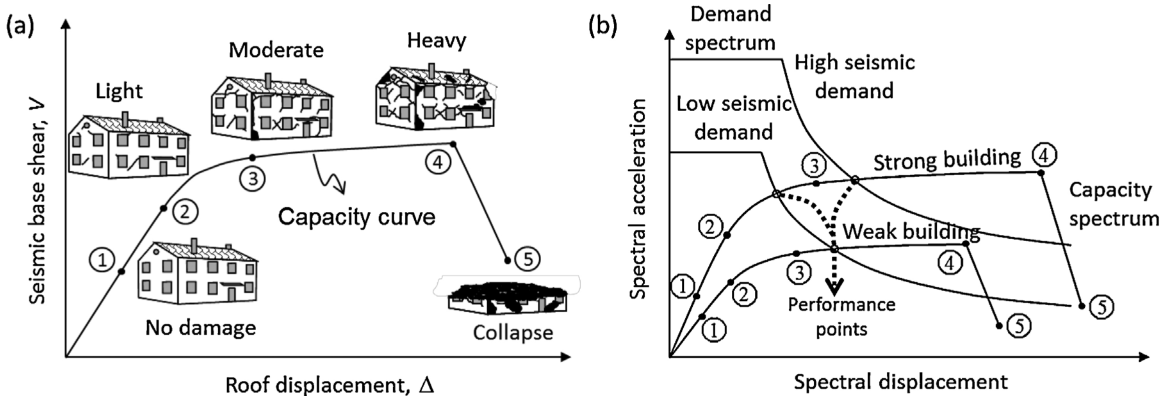


Fig. 12. Building capacity curve and damage states (a), and acceleration displacement response (ADRS) representation of seismic demand and capacity spectra.

comparison of seismic demand with structural capacity is through the Capacity Spectrum Method in the acceleration–displacement response spectrum (ADRS) format as shown in Fig. 12b (ATC, 1996, 2001; Freeman, 2004). A performance level is associated with the intersection of the capacity and demand curves, the so-called performance point. Fig. 12b illustrates the concept for two different levels of seismic demand and structural capacity.

Seismic performance evaluation according to TSC entails calculation of the deformation demands on members based on the structure's global response and estimation of member performance levels by comparison of these demands with the member capacity curves. Estimation of seismic performance for the building is based on the individual performances of members and the proportions of members with similar performance levels in the building. The building performance levels are designated as immediate occupancy (IO), life safety (LS), and pre-collapse (PC) performance levels (Fig. 13a).

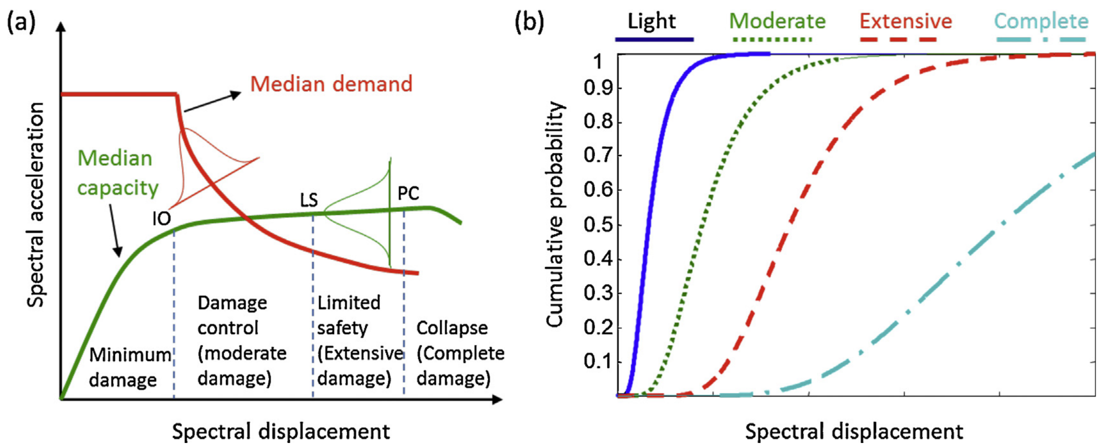


Fig. 13. Uncertainties in the demand and capacity spectra (a) and sample fragility curves for different damage states.

An additional concept which is not included in the TSC but is worth noting here is the probabilistic vulnerability assessment of buildings through fragility curves. The concept of seismic performance depicted in Fig. 12b naturally has uncertainties attached to both seismic demand and structural capacity components as illustrated in Fig. 13a. Depending on the level of these uncertainties, the location of the performance point may shift on the horizontal axis leading to different performance levels and corresponding damage states. This behavior can be used to construct the fragility curves shown in Fig. 13b that show the probabilities of different damage states ranging from light to complete (collapse) based on the spectral displacement demand on the structure. This concept lends itself for use in vulnerability assessment and loss estimation for individual as well as a stock of buildings.

An advantage of the fragility curves concept is that it can be based on rigorous analytical approaches (e.g. Singhal and Kiremidjian, 1996), empirical approaches (e.g. Rossetto and Elnashai, 2003; Straub and Kiureghian, 2008), hybrid approaches (e.g. Singhal and Kiremidjian, 1998; Akkar et al., 2005; Erberik, 2008) or just on expert opinion (ATC, 1985). Curves based on empirical data can be used to obtain rough estimates of seismic damage and loss while those based on actual nonlinear analyses can provide more reliable estimates of damage and loss for individual buildings. Depending on how rationally the fragility curves are obtained, the method can provide damage estimates within various uncertainty limits.

5. Rapid assessment and evaluation

The provisions in the TSC Ch. 7 for condition assessment and seismic performance evaluation of existing buildings somewhat represents the current state of the art and has the essential elements for satisfactory evaluation of buildings. In view of Turkey's grand challenge, however, the methodology described therein presents potential problems from the viewpoint of time it takes to complete the evaluation. From a professional perspective, implementation of the methodology from beginning to end for a building takes a small team formed by an engineer and technicians in the order of a month, which can only be shortened to a certain extent. Considering that millions of buildings are to be evaluated within the urban transformation initiative, there is an urgent need for rapid condition assessment and evaluation methodologies that enable rapid surveying and preliminary evaluation of buildings primarily for prioritization for further evaluation or to a lesser extent for decisions on retrofit or renewal.

Early methodologies for rapid screening and structural evaluation of buildings include FEMA 154 (ATC 1988) and ATC-14 (ATC, 1987), respectively. The former was intended to provide "a standard rapid visual screening procedure to identify, inventory, and rank buildings that are potentially seismically hazardous" (ATC, 2002) while the latter presented an evaluation procedure based on equivalent lateral force procedure and allowable stress design concept using capacity over demand ratios. FEMA 178 (BSSC, 1992) improved upon ATC-14 by using ultimate strength design principles in the evaluation. Later published FEMA 310 prestandard (ASCE, 1998) combined the basic philosophies of FEMA 154 and FEMA 178 as well as modern performance based evaluation concepts and presented an improved 3-tier evaluation procedure. This prestandard was replaced by the ASCE 31 standard (ASCE, 2003) which used the same evaluation philosophy and the tiered structure. ASCE 31 was later replaced by ASCE/SEI 41 (ASCE, 2014) which combined the evaluation and retrofit processes and included performance of nonstructural elements. It is worth emphasizing that FEMA 154 and ASCE/SEI 41's first tier evaluation aim at identification and ranking of buildings that are potentially seismically at risk and to quickly identify potential deficiencies. Performance evaluation of buildings is performed using more involved and reliable methodologies.

Particularly after the 1999 Kocaeli Earthquake, there has been an increased emphasis on the development of rapid screening and evaluation methods in Turkey. Inspired by the above-mentioned standards and notable earlier studies (Hassan and Sozen, 1997; Gulkan and Sozen, 1999), several rapid assessment and evaluation procedures were proposed in recent years (e.g. Yakut, 2004; Yakut et al., 2006; Bal et al., 2006; Sucuoglu et al., 2007). Most such methods are score-based empirical methods supported by statistical data, obtained from fairly large samples of buildings subjected to strong earthquakes, with limited or no consideration of in situ material properties. While these methods are important and needed for screening and prioritization of existing buildings for further evaluations, all too often there is a public tendency to accept or use the results of these methods as a measure of their buildings' seismic safety. It has been shown through research studies (Ozmen, 2013) and real life events that these methods cannot be used on their own for seismic safety evaluation of individual buildings.

In order to enable rapid identification of buildings under high seismic risk, a new specification was recently developed exclusively for use within the urban transformation initiative (RYTEIE, 2013). Following a brief transition period during which TSC Ch. 7 and RYTEIE were both in effect, this specification now governs the process of identification – but not performance evaluation or retrofitting – of existing buildings under seismic risk. Provisions of RYTEIE are based on those of TSC Ch. 7 with time saving simplifications. The specification also provides a score-based evaluation system for determining the regional distributions of buildings potentially under seismic risk.

Investigations for condition assessment are limited to the critical floor of buildings, defined as the lowest floor above ground or the soft story (one with a lateral stiffness considerably less than the others) if there exists one. Only two knowledge levels are defined as Limited and Comprehensive, the factors for which are specified as 0.9 and 1.0, respectively. For concrete strength, nondestructive tests are required on at least 10 columns or shear walls and minimum 5 cores are required from members yielding the lowest NDT results. The number of core samples is increased by one for each 80 m^2 of area in excess of 400 m^2. The in situ concrete strength is calculated as 85% of the mean concrete strength obtained from compression testing of cores. Steel reinforcement types and layout is to be determined in at least 20% of columns or shear walls, 10% of which is to be performed by exposing steel reinforcement while the remaining 10% can be inspected using covermeter. In situ yield strength of reinforcing steel is assumed as the minimum characteristic strength required by the code for the identified type of steel. If a local soil investigation is not performed, the local soil class is assumed to be Z4 which produces most unfavorable results.

Identification of seismic risk for buildings according to RYTEIE is based on linear elastic analysis using the base shear calculated from equivalent lateral force procedure. Proportion of members with demand to capacity ratios exceeding specified limits is used as the basis for seismic risk identification. As an indication of the risk level, a floor shear ratio is calculated for the critical floor calculating the total shear demand on columns and shear wall that are identified as at risk and dividing this value by the floor shear demand. This ratio can be used for ranking of evaluated buildings according to the level of seismic risk and their prioritization for further evaluation according to TSC Ch. 7.

A comparison of RYTEIE with TSC Ch. 7 reveals that the former offers some enabling tools for determining the regional distributions of potentially high risk buildings, risk identification of individual buildings based on relatively straightforward analyses, and ranking of buildings according to their approximate risk level. TSC Ch. 7 provides the next step procedure for seismic performance evaluation of identified buildings based on more rigorous analyses as a basis for retrofit or renewal decisions. When these specifications are compared from the perspective of time it takes to complete the respective assessment and evaluation procedures, however, the time savings offered by RYTEIE is not substantial. Both specifications inevitably use the essential components of condition assessment and structural evaluation such as material tests, structural information gathering, model generation, structural analysis and evaluation. Since the corresponding techniques and tools are common to the tasks in both specifications, it is unrealistic to expect drastic time savings in their implementation without improving the techniques and tools themselves.

6. Wishful thinking about an ideal methodology

In view of the presented grand challenge and the methodologies currently in use, some wishful thinking may be of help toward building a vision for an improved overall methodology that is rapid, reliable, economical, efficient, tractable, and scalable. In the author's opinion, characteristics of an ideal methodology include, but are not limited to, the following:

- Adopts a probabilistic framework and a staged assessment and evaluation methodology that updates the reliability of evaluation at each stage.
- Allows rapid condition assessment and evaluation; producing the preliminary evaluation report after the first site visit, with a quantitative assessment of the associated uncertainty limits.
- Makes optimum use of technologically advanced tools that can increase the speed of assessment and evaluation.
- Information gathered or produced at each stage lends itself for use in the remaining stages.
- Allows implementation of different rapid evaluation methodologies for consideration of interested parties.
- Includes rational assessment and evaluation procedures based on science and mechanics of materials and structures, particularly in later stages of evaluation.
- Allows performance based evaluation.
- Reliability of evaluation is continuously updated as more or higher quality information becomes available.
- Building information is organized in a structure that can be stored and maintained in a central database and can be easily accessed by authorized parties.
- Includes geographical information systems (GIS) support for assessment of regional distribution and beyond.
- Through database structure and GIS support, allows instantaneous reporting of building inventory, descriptive statistics, risk and loss estimations based on statistical or performance evaluations.
- Can be used for preparedness and seismic hazard mitigation before an earthquake and for rapid damage assessment and recovery after.

A methodology that embodies all the characteristics listed above may be difficult to achieve in the short term, but it is important to note that no single list item is far-fetched in view of the current state of the art. A wise strategy may be to put emphasis on the new or improved techniques and technologies that can potentially speed up the assessment and evaluation process without significantly compromising reliability.

7. Promising methods and technologies

If a time chain of events is considered in condition assessment and evaluation of existing buildings, one immediately notices that certain components of the investigations are particularly time consuming. Considering the methods of condition assessment shown in Fig. 11, for instance, it is easily noticed that the results of all but concrete core tests are available at the end of the site investigations. All major current codes including those in Turkey require concrete core sampling and compression testing as the single most reliable method of in situ concrete strength assessment. Use of nondestructive testing methods for the same purpose is allowed only when correlated with concrete core tests.

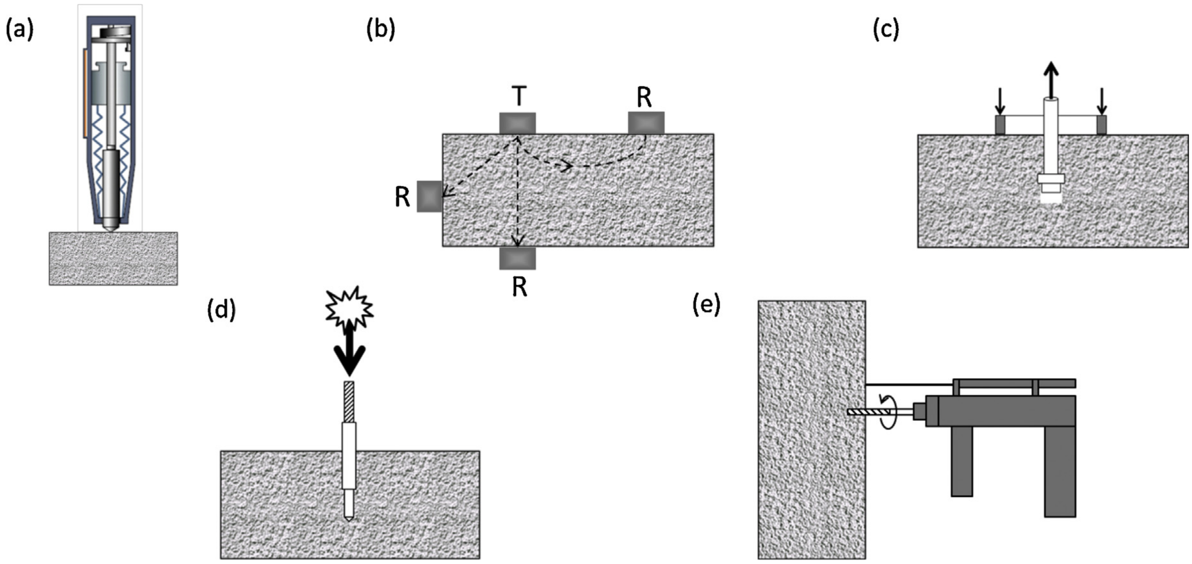


Fig. 14. NDT techniques that have been or potentially can be used for in situ strength assessment of concrete: (a) rebound (Schmidt) hammer (RH); (b) ultrasonic pulse velocity (UPV); (c) pull-out force; (d) penetration resistance; (e) drilling resistance.

Various NDT methods that have been, or potentially can be, used for in situ strength assessment of concrete are shown in Fig. 14. The rebound hammer technique, generally called the Schmidt hammer in attribution to its inventor, is essentially a surface hardness method that can be indirectly correlated to concrete strength. This is by far the most commonly, and too often wrongly, used NDT method in Turkey and abroad. The ultrasonic pulse velocity (UPV) method makes use of the velocity of ultrasonic pulses obtained from direct (opposite faces), semi-direct (neighboring faces) and indirect (same face) arrangement of the transmitter (T) and receiver (R) transducers (Fig. 14b) to estimate concrete strength. The pull-out force method is a mechanical technique that measures the force needed to pull-out a rod embedded in concrete before or after hardening and correlates it to concrete strength. The penetration resistance technique involves driving a steel probe or pin into concrete using a powder actuated driver that delivers a known amount of energy. The measured penetration depth or the exposed length of the probe or pin is related to concrete strength. The drilling resistance method (Fig. 14e) is not yet an established method of concrete NDT but is included here for its high potential for use in strength assessment of concrete in a rapid and reliable fashion. The method simply involves estimation of concrete strength based on the ease of drilling a hole inside concrete.

TS EN 13791 provides a framework for use of the methods shown in Fig. 14a–c for in situ strength assessment of concrete in conjunction with concrete core tests. TS 13543 (2013), a recently developed umbrella standard, which is a compilation from international standards and other resources, provides an extensive list of NDT techniques applicable to reinforced concrete structures including those in Fig. 14a–d with references to specific standards and resources. The drilling resistance method shown in Fig. 14e is currently a topic of research and development (Pamplona et al., 2007; Felicetti, 2006, 2011).

Ongoing research on nondestructive evaluation of concrete strength (e.g. within RILEM TC 249-ISC Non Destructive In Situ Strength Assessment of Concrete) is looking into ways of improving existing methods and developing new ones with particular emphasis on combined use of techniques to improve the accuracy and reliability of results (Breysse, 2012a,b). Progress in this area, especially in techniques that have a mechanical component such as those in Fig. 14c–e, combined with parametric and probabilistic studies investigating the sensitivity of seismic performance evaluation on the uncertainty limits of material properties, may allow replacing core tests with individual or combined NDT techniques in the near future.

Seismic performance evaluation of buildings requires linear or nonlinear structural analyses which ideally must be based on accurate knowledge of the building's structural system and member details. Gathering this information becomes particularly difficult and time-consuming when the structural drawings do not exist or do not match the existing structure, which is too often the case in Turkey. Recognizing that the worst structural model based on the actual building is better than having no model at all, a staged approach that makes use of advanced technologies can be adopted. Photogrammetry, mobile technology and augmented reality can be particularly useful for obtaining a quick structural representation of the building including the geometric dimensions of the building, the structural grid, and individual members (e.g. Kang and Wang, 2013). Knowledge of the structural details can be built incrementally, starting with the findings from condition assessment investigations (see Fig. 11), making evidential deductions such as giving the structure credit for standing up and having resisted previous seismic events, and making conservative assumptions to be updated if the determined seismic performance of the building, being unsatisfactory, warrants more reliable structural data from additional on-site investigations.

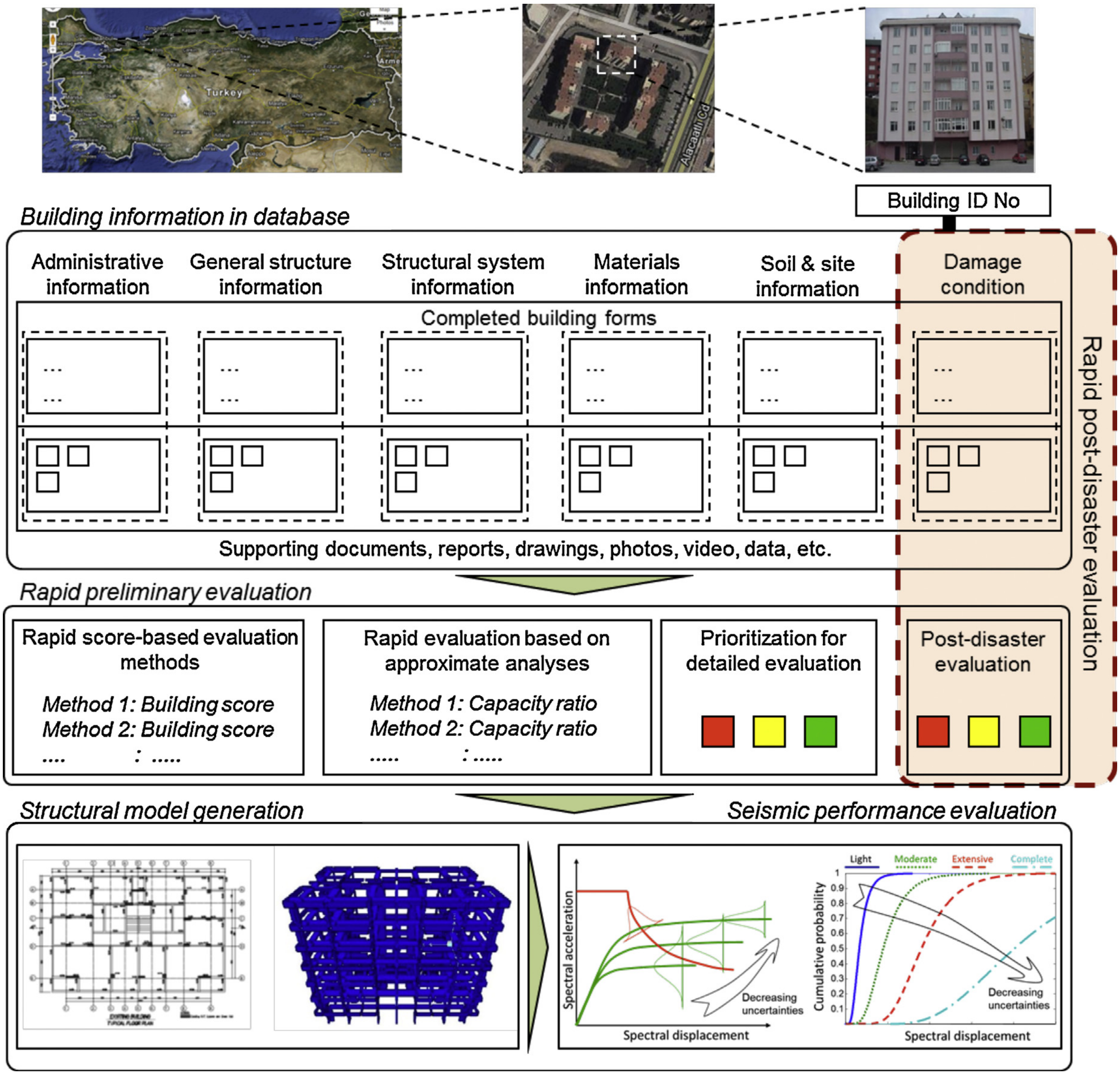


Fig. 15. A possible framework methodology for condition assessment and seismic performance evaluation of existing buildings.

Several types of metrics can be used as an indication of seismic performance depending on the evaluation methodology. Examples of these used in Turkey and elsewhere are presented in Sections 4 and 5, ranging from structural scores obtained from preliminary surveys to performance levels obtained from pushover analysis and capacity spectrum method. One metric that appears to be applicable to all different approaches is the concept of fragility curves discussed in Section 4 (see Fig. 13). Depending on the amount and reliability of building information, the basis of fragility curves can range from expert opinion to rigorously developed analytical curves, providing a measure of performance in a similar form for all cases, but with different uncertainty limits. Seismic performance indicators in the form of fragility curves may also be a more realistic and honest representation of seismic performance from the building owners' viewpoint since the real information sought by the owner is the level of expected damage, particularly of complete damage (collapse), which always has a probability of occurrence however small it may be. It is also of interest to see how the probabilities of each damage state change with additional investigations or mitigation actions. The use of fragility curves can be further enhanced by introducing time-dependent effects of aging and deterioration, which constitutes an exciting area of future research and development (Ghosh and Padgett, 2010).

Potential improvements to the current state of the art are certainly not limited to the areas discussed above. Strategic development and implementation of improvements prioritized with respect to impact on the speed and reliability of evaluations appears to be the optimum approach. Fig. 15 presents a possible framework methodology that includes GIS and database support and has pre- and post-disaster implementation potential. Elaboration on these technologies and their implementation is left to a future publication due to space restrictions.

8. Conclusion

Turkey's grand challenge for the next two decades is introduced with related statistical data, information on the current state of the art, and possible improvements through use of efficient techniques and new technology. It is argued that the more sensible approach to rapid evaluation of buildings is not consorting to empirical half measures but rather developing an integrated incremental methodology that favors mechanics based assessment and evaluation methods to empirical ones, expedited by use of technology and enriched by rational assessment of evaluation uncertainty. From an engineering perspective, this massive initiative represents a unique problem and presents motivation and opportunities for significant progress that may build capacity and competitiveness for Turkey and Europe.

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