Assessment of Buildings in Turkey for Earthquake: *Is there another way?*

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2009 WCCE-ECCE-TCCE
Joint Conference

ABSTRACT

This paper investigates whether the approach adopted in New Zealand for the assessment of buildings for earthquake might yield benefits in the earthquake assessment of existing buildings in Turkey.

A comparison is made between the approach adopted to assess and retrofit school buildings as part of one of the Istanbul Project Coordination Unit's ISMEP projects for earthquake strengthening of school buildings and the methodology currently recommended in New Zealand for assessing New Zealand buildings.

The comparison has been made by way of example using one of the buildings from that project which has been subjected to assessment using both the linear elastic method (as specified in the current Turkish Earthquake Code) and the inelastic pushover method.

The results would indicate that the NZ displacement-based assessment method predicts a higher level of seismic performance for this particular building than is predicted by the methods currently being used in Turkey. This is achieved using simple hand methods and the NZ method of assessing member ductile capacity.

INTRODUCTION

Starting in late 2007 and continuing through 2008, Turkish consultant Prota Design Engineering and Consultancy Ltd (Prota), supported by specialist sub consultant Beca International Consultants (Beca) assessed 241 school buildings within the Municipality of Istanbul for earthquake. When necessary, retrofit schemes were developed to meet minimum performance requirements.

The work was undertaken for the Istanbul Project Coordination Unit (IPCU) as part of the Istanbul Emergency Mitigation Project's (ISMEP) *CB1.3/D Consultancy Services for Retrofitting Designs of Selected Public Buildings in Istanbul*.

The assessment methodology adopted was developed by Prota (in close cooperation with the IPCU). This methodology was very similar to that presented in the *Guidelines for Seismic Retrofitting of School and Hospital Facilities in Istanbul* issued in draft by the IPCU in May 2008.

In this paper the results from the methodology adopted during the above project are compared with those arising from the assessment processes presented in the New Zealand Society for

Earthquake Engineering (NZSEE) study group guideline document *Assessment and Improvement of the Structural Performance of Buildings in Earthquake*[1].

The NZSEE guideline document presents both force and displacement based methods of assessment. The displacement based methodology has been used for the purposes of the comparison outlined in this paper. It is recognized that displacement based methods for assessment and design are currently under significant development and procedures have progressed further in the two years since the NZSEE guidelines were published.

The NZSEE guideline is the authoritative assessment document in New Zealand and this has been acknowledged by the New Zealand Department of Building and Housing

The intention of this paper is not to criticize the Prota/IPCU methodology but rather to suggest that there may be benefit in also considering other approaches.

EXAMPLE BUILDING

The example building is representative of a number contained within the inventory of school buildings considered in the investigation. The design details vary slightly from building to building within this grouping but to all intents this can be considered a standard design. This particular building was constructed in 1993.

Refer Figures 1&2. The building consists of two parts; the main section, approximately 40m x 19m, abuts a smaller 4.5m x 27m stair tower structure on one end (left hand of Figure 1). It is well separated from adjacent buildings but there is only a small, 5cm, separation between the two parts.

Figure 1: Front Exterior View of Example Building

The structure of the building is a two way three storey reinforced concrete frame with poured in-situ concrete floor and roof slabs. The frame of the main section of the building is supplemented by reinforced concrete shear walls in both principal directions, two at each end and one on each side of the internal stair well approximately mid way between, and four located on either side of the central corridor orientated in the longitudinal direction. The stair tower has two reinforced concrete shear walls supplementing the lateral capacity in its longitudinal direction which is perpendicular to the longitudinal axis of the main section of the building.

Non-structural partitions are typically of 150mm thick rendered unreinforced clay masonry located on frame lines. On each side of the main entrance the partition thickness increases to 300mm. These partitions form infill panels within the frame and therefore provide some lateral capacity. The partitions are reasonably uniformly spread over the plan and height of the building. The spandrel panels under the windows on the exterior faces of the building are also constructed from unreinforced masonry.

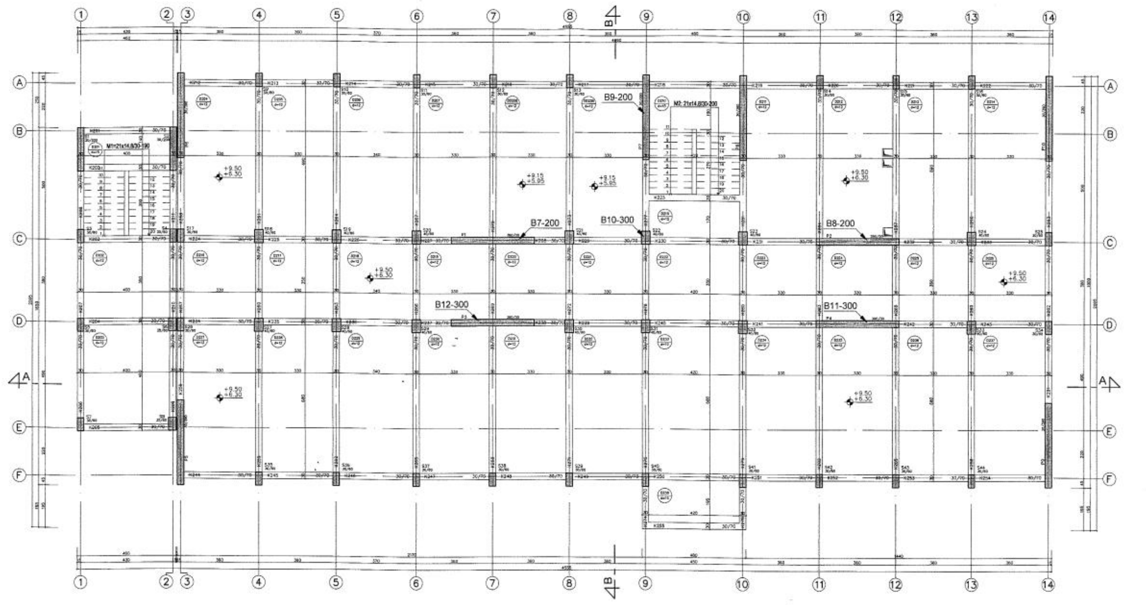

Figure 2: Typical Plan of Example Building Showing Frame and Wall Layout

The example building is located on stiff soil (Z2 as defined in the Turkish Earthquake Code (TEC)[2]). The foundation consists of a grillage of 1m deep reinforced concrete ground beams located on the line of the frames and continuous under both parts of the building. Excavations revealed that the perimeter and longitudinal (under the corridor frames) beams are flanged to provide additional bearing.

Reinforcing details for the structure above ground level were confirmed by Ferroscan to be as shown on the original construction drawings. Construction details and reinforcing contents of the foundation are not available and were inferred from excavations and reference to what was considered typical reinforcing detailing of the time.

Cores taken from the concrete elements within the building gave concrete compression results ranging from 11 to 24MPa with an average of 18MPa and σ of 3MPa. Mean minus 1σ of these results gives 16MPa and this was assumed as the representative concrete compression strength for the building for all of the calculations reported below. A material coefficient of 1 was used for all assessments.

Testing of a sample of reinforcing steel taken from the building indicated a yield stress of 220MPa. The bars for main and stirrup reinforcement are un-deformed. Beam stirrup and column tie reinforcement is terminated with 135 degree hooks.

The overall impression is that the building is well configured for good performance in earthquake notwithstanding the relatively mediocre concrete strength. As will be confirmed below, the gap between the two parts of the building is insufficient to prevent the parts coming into contact during the earthquake scenario event described below. Therefore, for the purposes of the following discussion, it has been assumed that the two parts will be connected together to remove this deficiency.

EARTHQUAKE SCENARIOS

The tectonics of the Marmara region is dominated by the North Anatolian Fault (NAF). Consideration of historical earthquake activity and the tectonics would advocate that the most likely earthquake to affect Istanbul would be a M_w 7.5 event occurring on the segments of the NAF closest to Istanbul. Studies by Parsons et al (2000)[(3)] have estimated the probability of occurrence in the next 30 years of a magnitude 7 or greater earthquake in the Marmara region targeting İstanbul as 62 ± 15 percent.

The earthquake shaking assumed to be representative of this scenario event, and used for the calculations outlined below is shown in Figure 3. This acceleration response spectrum has been derived by averaging the mean plus 1σ estimates obtained from the Akkar and Bommer (2007)[(4)] attenuation relationship and the 2475 return period spectrum obtained from the TEC for the example building location (Zone 2) and Z2 subsoil classification.

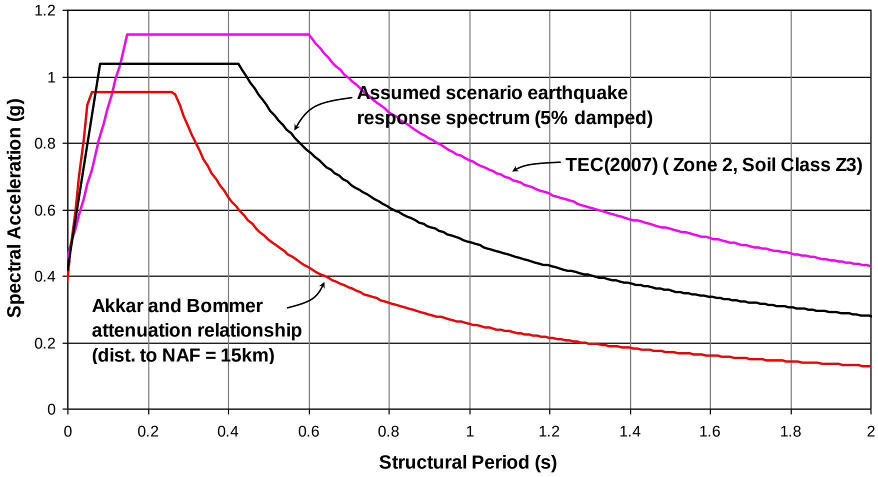

Figure 3: Scenario Earthquake Acceleration Spectrum

It is apparent from Figure 3 that, for the example building, the resulting scenario spectrum is conservative when compared with the Akkar and Bommer prediction.

The corresponding Acceleration-Displacement Spectra for various damping levels derived in accordance with the NZSEE guidelines is shown in Figure 4.

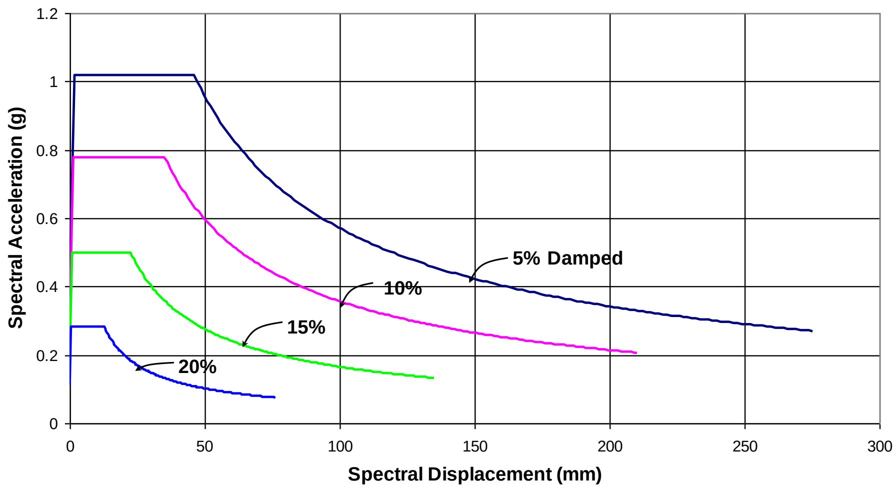

Figure 4: Scenario Earthquake Acceleration-Displacement Spectra for Various Levels of Damping

A smaller earthquake scenario corresponding to a M_w 6.5 event occurring on the segments of the NAF closest to Istanbul was also derived for checks against immediate occupancy objectives. Checks for this objective were not critical for the assessment of this building so have not been considered further in this comparison.

LINEAR ELASTIC METHOD

One of the methods of assessment adopted for this building for the CB1.3/D project was based around the results of a linear elastic analysis carried out in accordance with Chapter 7 of TEC (2007).

A computer model of the building was created with the following features;

- Member stiffnesses of 40%Igross and 65%Igross for beams and shear walls, and columns respectively to allow for cracking.
- No accidental mass eccentricity.
- Rigid end zones to model joints between members
- Soil structure interaction ignored.
- Fixed support condition assumed for bottom joints of basement floor columns and shear walls.

Structural analysis of the building was carried out using the Probina computer structural analysis package. A modal analysis was completed, fundamental periods for each direction of loading determined and the actions determined in all structural elements for the scenario earthquake in accordance with the requirements of the TEC. The resulting natural periods of vibration were determined to be 0.52 and 0.44 seconds, for the longitudinal and transverse directions respectively.

The extent of confinement of the reinforced concrete element sections were determined and capacities calculated according to the ultimate limit state methods contained within the Turkish Concrete Design Standard[5].

Demand Capacity ratios were determined for each member and the resulting damage limit state evaluated in accordance with the capacity control method contained in TEC.

The results are summarised in Table 1.

Direction	Storey	% Beams in severe damage limit state	% shear carried by columns and walls in severe damage limit state	Storey drift	% Base shear carried by walls
+Transverse	Basement				86%
	Grd	19	83	0.003	
	1st	19	2	0.006	
	2nd	19	14	0.007	
	3rd	19	69	0.006	
	4th				
-Transverse	Basement				86%
	Grd	19	83	0.003	
	1st	19	3	0.006	
	2nd	19	12	0.007	
	3rd	19	67	0.006	
	4th				
+Longitudinal	Basement				84%
	Grd	22	82	0.003	
	1st	22	12	0.006	
	2nd	22	4	0.006	
	3rd	17	51	0.005	
	4th				
-Longitudinal	Basement				84%
	Grd	17	82	0.003	
	1st	28	23	0.005	
	2nd	28	8	0.006	
	3rd	17	45	0.005	
	4th				

Table 1: Summary of Results from Linear Elastic Method

If any one of the following criteria are exceeded, retrofit is considered necessary;

- % Beams in severe damage limit state; 30%, unless at least 75% shear is carried by walls
- % Shear carried by columns and walls in severe damage limit state: 40% top floor, 20% elsewhere
- Storey drift: .03

It is apparent from the results presented in Table 1 that the % of shear carried by columns and walls that are in the severe damage limit state exceeds the specified limiting value and therefore the linear elastic method determines that the building does not meet the acceptance criteria. The building therefore requires seismic retrofit in both directions.

INELASTIC PUSHOVER ANALYSES

Inelastic pushover analyses were carried out using the SAP 2000 computer package. Inelastic properties were determined from moment curvature analyses completed using Probina. Target displacements appropriate for the scenario earthquake defined above were calculated for each direction to the requirements of TEC.

The results are shown in Figures 5 and 6 and confirm that retrofitting would be required using this procedure.

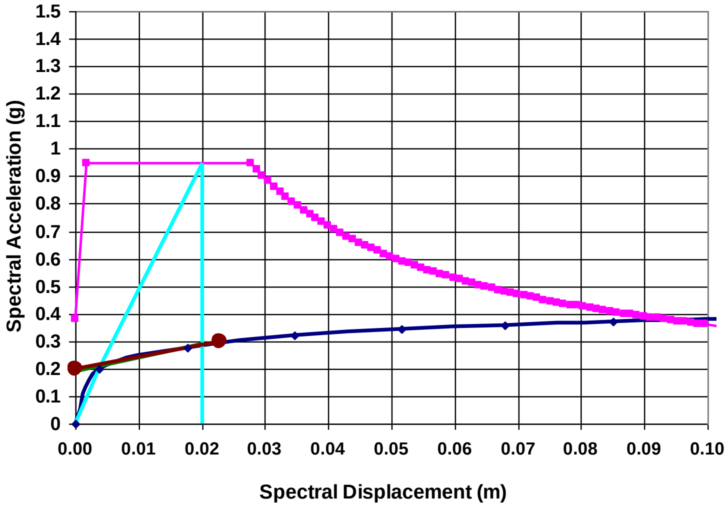

Figure 5: Results From Inelastic Pushover Analyses – Longitudinal Direction

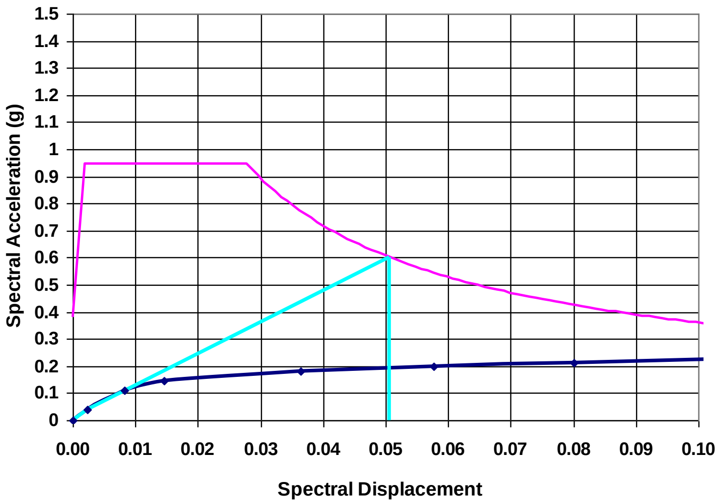

Figure 6: Results From Inelastic Pushover Analyses – Transverse Direction

THE NEW ZEALAND APPROACH

The New Zealand approach adopted for the purposes of this comparison is the displacement based method set out in the NZSEE guideline document, and in particular the method described for dual frame-wall buildings.

Displacement based methods place a direct emphasis on establishing the ultimate displacement capacity of the lateral force resisting elements and, where applicable, the non structural elements.

The procedures encompassing this approach represent a relatively recent development and can be expected to be fine tuned as more experience is gained in applying the method to real structures.

The key steps adopted in this investigation can be summarised as follows;

1. Evaluate the probable seismic strength of the beams, columns and walls.
2. Determine the post-elastic deformation mechanism of the structure and hence the probable horizontal base shear capacity of the structure. In this investigation the Simple Lateral Mechanism Analysis (SLaMA) Procedure outlined in the NZSEE guidelines was used. This is a hand method based on substitute, equivalent single degree of freedom structures where the stiffness and capacity (strength, yield and plastic displacement) of frames and walls are evaluated separately using simple rules and then combined to give an estimate of the total force displacement curve for the dual system. The displacement capacity of the structure is the lower of that determined for either the frames or walls. The method includes checks for beam or column hinging in the frames, the potential for premature shear failures in the members and the influence of curvature ductility on shear capacity. For this building the frames are found to exhibit a yielding beam mechanism.
3. Determine the equivalent viscous damping available from the structure at a displacement approaching the capacity.
4. Compare the capacity determined above with the demand represented by the appropriately damped acceleration-displacement spectrum.

The SLaMA method is reasonably simple and quick to apply and provides a good understanding of how a building resists lateral loads.

RESULTS FROM THE NEW ZEALAND APPROACH

The results determined from application of the New Zealand approach are shown in Figures 7 and 8.

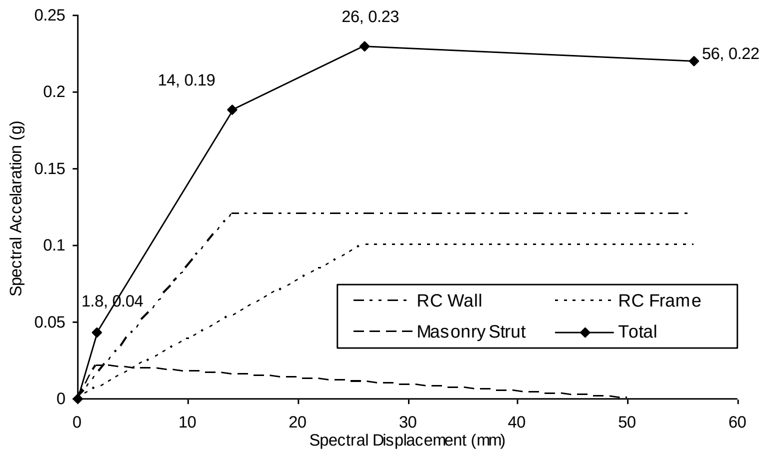

Figure 7: Load Deflection Curve using NZ Approach – Longitudinal Direction

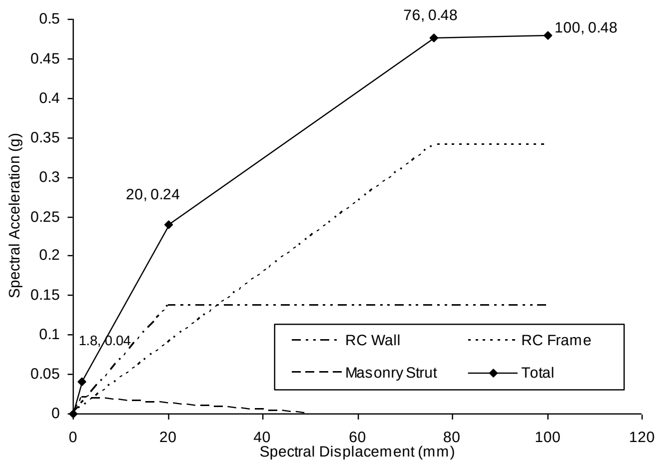

Figure 8: Load Deflection Curve using NZ Approach – Transverse Direction

The effects of the frames, concrete walls and unreinforced infill panels have been included. The deformation relationship for the infill panels has been estimated from the stress strain relationships proposed by Binici and Ozcebe(6).

The displacement capacity of the structure is limited by the capacity of the walls (assuming that failure of the infill panels can be tolerated)..

Figure 9 shows the load deflection capacity curve compared with the demand curves presented in Figure 4. The NZSEE guidelines would suggest available effective hysteretic damping between 10 and 15% and approximately 17% in the longitudinal and transverse directions respectively. The NZ approach would therefore suggest that retrofitting was not required for either direction.

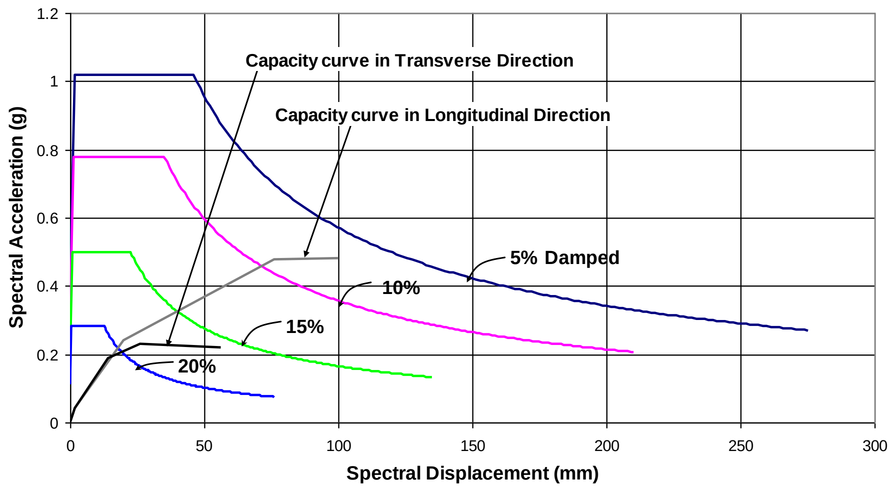

Figure 9: Capacity and Demand Compared for the NZ Approach

CONCLUSIONS

The conclusions can be summarised for the example building as follows;

- The shape of the load deflection curve predicted by the NZ SLaMA method compares well with the results from the inelastic pushover analyses.
- The plastic displacement capacities predicted by the NZ approach exceed those calculated from the TEC. The NZ approach is therefore more optimistic than TEC.
- The NZ approach suggests that the demand can be significantly reduced if the effective hysteretic damping is accounted for.
- The NZ approach would indicate that no retrofit is required for this building in order to satisfy the demand from the scenario earthquake.
- There would be value in further investigating the differences between the approaches used for assessment in Turkey and NZ to see if the more optimistic results implied by the NZ method can be justified for buildings in Turkey.

ACKNOWLEDGEMENTS

The authors acknowledge the contributions made by the teams in their respective organisations in Istanbul, Ankara and Wellington, NZ in regard both to the CB1.3/D project and the comparisons made as part of this paper. We also acknowledge the inputs made by Prof. Haluk Sucuoğlu and Doctor Sinan Akkar from the Middle Eastern Technical University in Ankara who were the Approval Authority for CB1.3/D.

A special tribute is made to the late Professor Bob Park (University of Canterbury) and Professors Emeritus Tom Paulay (University of Canterbury) and Nigel Priestley (University of California San Diego) for their significant contributions to the sections of the NZSEE guideline document that were used during this exercise.

REFERENCES

1. NZSEE
2. TEC
3. Parsons et al (2000)

4. Akkar and Bommer
5. Turkish Concrete Design Standard
6. Binici and Ozcebe