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PERFORMANCE BASED SEISMIC DESIGN OF SOYAK CRYSTAL TOWER - GETTING A SAFER AND MORE ECONOMICAL STRUCTURE

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ABSTRACT

Soyak Crystal Tower is a signature highrise office building in Levent, Istanbul. The structure of the tower is reinforced concrete, with the last occupied floor at 140 m above ground and the top of the tower at 168 m above ground. Because of its highly seismic location, the structure has a high likelihood of experiencing a strong earthquake in its lifetime. Although this tower could be designed using the standard force-based design approach, it was decided to use the state of the art seismic performance based design (PBD) approach. This paper will discuss the reasons for choosing performance based design, the design process including how some complex design questions were resolved, and the advantages gained by this approach.

The main lateral load resisting system of the structure is the concrete shear wall core. Although many seismic design codes requires the presence of a moment frame system that can carry part of the total lateral loads for a structure of this height, seismic performance based design was used to show that the structure meets the required seismic performance criteria without relying on moment frames. Removing the moment frame design requirement provided a chance for significant reduction in cost of the structure without adversely affecting its performance. It also provided more flexibility for architectural and aesthetic design of the tower.

In additions, the geometry of Crystal Tower posed unique challenges. The columns twist and turn, merge, bifurcate, and change direction throughout the height to conform to the unique beautiful contours of the building. However, these variations in geometry produce large horizontal kick forces that have to be transferred back to the concrete core of the structure. The amplitude of these kick forces depends on the axial forces of the columns, including seismic forces. A force-based design per code with reduction factors applied to the elastic demands would significantly underestimate these forces and result in an underdesign, affecting the stability of the gravity load path as well as the lateral load path in a seismic event. Seismic PBD allowed for realistic estimation of seismic forces and hence, a safe reliable design. Furthermore, it was decided that the building performance in a seismic event should be superior to that of similar structures. Seismic PBD allowed for the higher performance targets to be directly incorporated into the design and verified by analysis.

By conducting performance based design, we were able to achieve several important goals. The structure was designed for a superior seismic performance, and the desired performance was explicitly verified. By modelling the actual nonlinear behaviour of the structure instead of relying on reduced elastic forces, we achieved a more optimal design that provides strength and stiffness where it actually helps the seismic performance. The elimination of the moment frame requirements resulted in

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significant reduction in the cost of the framing and elimination of certain interior and exterior frames altogether, with added benefits in flexibility for architectural and mechanical design of the building, increased useable floor heights, more transparent lobby space, and a more aesthetic look for the building. In summary, performance based design of the iconic Crystal Tower allowed for a more reliable, efficient, and economical design that exceeds the seismic performance of similar buildings while giving the rest of the design team more flexibility to design a beautiful welcoming environment for the occupants of the building.

INTRODUCTION

Soyak Crystal Tower is a signature highrise office building in Istanbul. Located about 25 Km from Marmara fault, proper seismic design was very important to the structural stability of the tower. The building has 35 stories above ground and 8 basement levels, with the last occupied floor space at 140 m above ground and the top of the tower at 168 m above ground. The construction of the structure and its envelop has been completed (Figure 1).

The main lateral load resisting system of the structure is the concrete shear wall core. For a building of this height, many design codes require the presence of a moment frame system that can carry part of the total lateral loads (ASCE, 2010). Because of the high significance assigned to this structure by the client, it was required that it meet American design standards as well as Turkish standards. This would mean that the moment frame in both directions would have to be designed to carry 25% of the total lateral forces. This task would be particularly challenging in the north-south direction (Y direction in the models). The exterior framing in this direction is highly twisted and unable to contribute much to lateral resistance (Figure 2). Therefore, additional interior framing in the Y direction had to be added to the structure to carry the 25% of lateral loads. However, it is understood based on the different deformed shape of a shear wall core and a moment frame system that the moment frame system does not provide much benefit to the combined system (Smyth and Coull, 1991, Taranath, 2005). Seismic performance based design allows for a design using a core only lateral system, as long as it is shown that the structure meets the required seismic performance criteria. The chance to remove the moment frame requirements, resulting in significant reduction in cost of the structure, provided strong incentive for performance based design. In addition, this would allow for significant aesthetic and usability gains. For example, it would allow for removing perimeter framing at the beautiful 12 m high lobby of the structure, enhancing its beauty and sense of openness. In addition, removing much of the interior framing and reducing the size of the rest of the beams allowed for higher useable ceiling heights, a significant benefit to the building.

There were also serious structural reliability concerns that would be hard to resolve with a standard force-based design per code. The standard design is based on results of elastic analysis of the structure and reducing those forces by a large factor to account for the expected nonlinear behaviour of the structure. However, nonlinear seismic behaviour of a structure changes its distribution of forces and displacements. It is known that the actual forces in the structure will be larger than the nominal design forces. In standard design for highly seismic areas, capacity design approach is used depending on the particular lateral system of the structure to ensure ductile behaviour and lateral stability. However, the geometry of Crystal Tower posed unique challenges to following this approach. The columns twist and turn, merge, bifurcate, and change direction throughout the height to conform to the unique beautiful contours of the structure. However, these variations in geometry produce large horizontal kick forces that have to be transferred back to the concrete core of the structure. Figure 3 shows three of the east-west frames of the structure. Locations where change of slope causes horizontal kick forces have been identified. The amplitude of these kick forces depends on the axial forces of the columns, including seismic forces. These seismic forces would be underestimated by an unknown amount in a standard design. A failure to provide for the adequate horizontal load path for these kick forces would affect the stability of the gravity load path as well as the lateral load path in a seismic event. Seismic PBD would allow for realistic estimation of seismic forces and hence, a safe reliable design.

Furthermore, it was decided that the building performance in a seismic event should be superior to what is expected of a similar structure in standard design. The force-based standard design allows

for increasing the design forces for buildings of higher importance, with the assumption that this will lead to better seismic performance. However, this approach does not lend itself to an explicit verification that the building will indeed meet the assumed performance goals. Seismic PBD allows for the higher performance targets to be directly incorporated into the design and verified by analysis.

As a result, it was decided to proceed with seismic performance based design for the Crystal Tower with a core-only lateral system. This paper discusses the general seismic PBD approach. It will also discuss some key features of the design, including the proper design of horizontal load path at locations of column kick forces.

GENERAL PERFORMANCE BASED DESIGN APPROACH

Performance-based design relies on realistic evaluation of the seismic behaviour of the building subject to site-specific ground motion. The performance is typically checked at multiple levels of ground motion. The aim is to model the actual forces and deflections caused by the earthquake and verify the performance of the structure and its components. As such, best estimate mass and material properties are used in the dynamic analysis. Furthermore, different actions are divided into two sub-categories based on the expected failure mode: force-controlled and deformation-controlled. Deformation-controlled actions are ductile and therefore, expected material properties are used for checking their performance. Force-controlled actions are brittle and therefore, lower bound specified material properties are used for checking their performance.

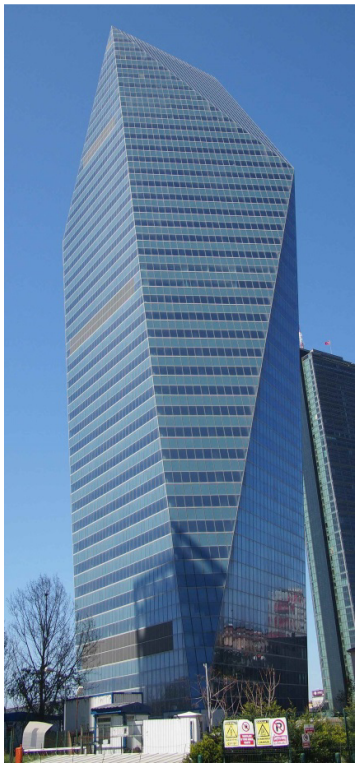

Figure 1. View of Crystal Tower

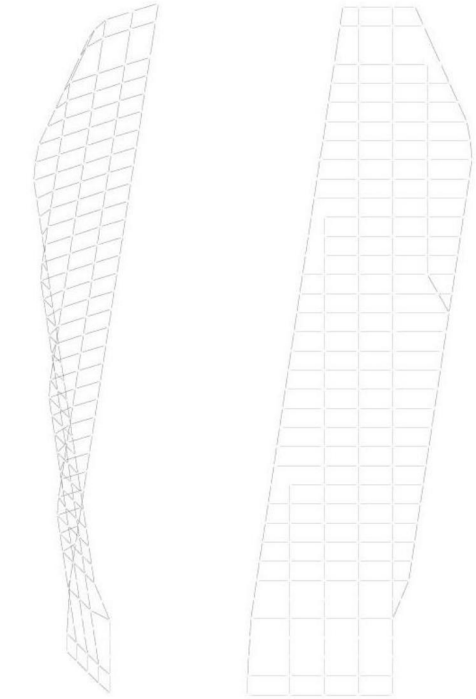

Figure 2. Building exterior frames. The other two faces were mirror views of these frames.

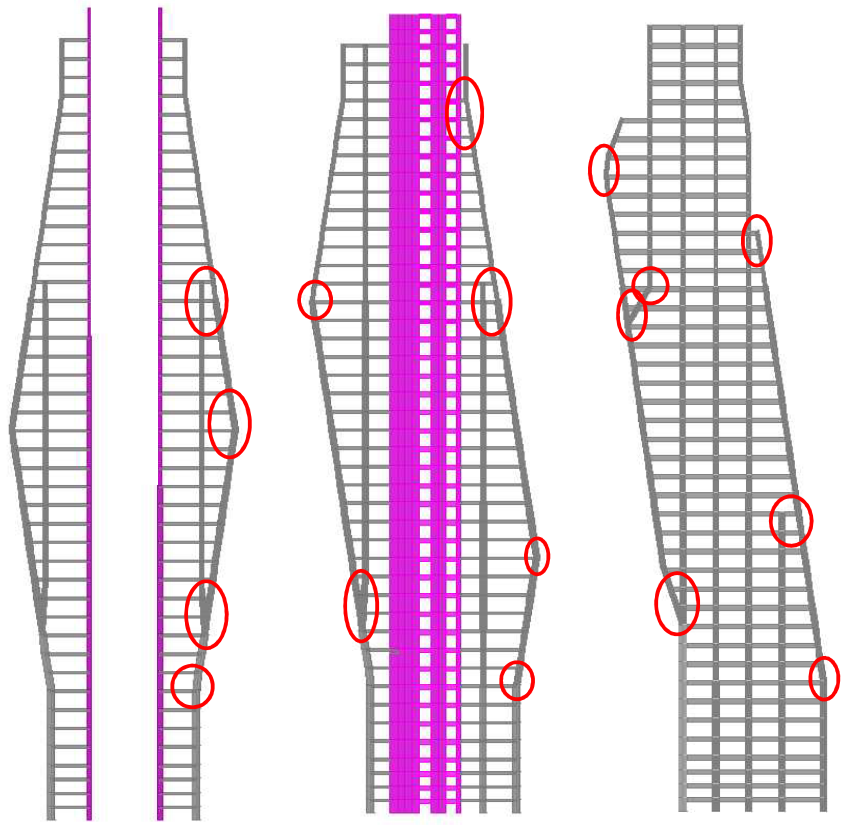

Figure 3. Areas of concern, including location of horizontal kick forces, as well as very short beams in the initial geometry that were susceptible to concentrated damage because of their depth to length ratio. The geometry was modified as necessary.

There are several documents available internationally that provide guidelines for seismic PBD. The most commonly referenced document was ASCE 41-06 (ASCE 2006), supplemented in 2007 (ASCE, 2007), which was selected as the basis document for PBD. However, for material overstrength factors (to obtain expected concrete and rebar properties), suggested values from draft local guidelines for Istanbul were used as they were thought to be more reflective of local available materials (IMM, 2008). ASCE 41 defines three performance levels: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). Roughly speaking, Immediate Occupancy represents a structure that is just slightly above the linear range of its response, with limited minor cracking of structural components. Life Safety represents a structure well into nonlinear range of its response, with considerable damage and potential permanent drifts that nonetheless has some residual strength and stiffness and a good margin against collapse. Collapse Prevention represents a performance in which the structure has experienced significant damage and is near collapse. For a typical building, the recommendations are IO performance for a frequent seismic event (such as 50 year return period), LS for design level earthquake (typically 475 year return period) and CP for the maximum considered earthquake (2475 year return period). The ground motions and response spectra corresponding to these return periods were provided by the experts in form of seven pairs of ground motions for each seismic level, along with the associated response spectra (Erdik et al., 2006).

Table 1 shows the modelling assumptions and target performance for this design. It is important to note that the target performance for the Crystal Tower is superior to that of similar buildings. At DBE-1, the structure is intended to meet Immediate Occupancy requirements, which imply minimal damage to the structure. As such, linear analysis would be appropriate at this seismic level. ASCE 41 procedures allow for linear analysis in this context, with the unreduced forces checked against m-factor times the capacity of members under different actions. Unlike the standard code based design where there is one constant reduction factor for the seismic forces, the m-factor which

effectively has the same function varies by the type of action. More ductile actions warrant higher m factors. In general, these m factors are smaller than the standard force reduction factors in code. At DBE-2, the target performance is Life Safety, which implies significant nonlinearity. Nonlinear analysis would be appropriate at this seismic level.

Table 1- Assumptions for Modeling and Performance Evaluations

Ground Motion Intensity	Ground Motion Return Period	Type of Analysis	Modelling Software	Target Performance	Force Reduction Factor	Material Model Assumptions
DBE-1	475 years	Linear Response Spectrum	ETABS	Immediate Occupancy	1.0	Expected (mean) properties
DBE-2	2475 years	Nonlinear Dynamic Analysis	PERFORM 3D	Life Safety	Not Applicable	Expected (mean) properties

The next step in the design process was an initial design, whose performance was to be verified and modified through the PBD process. The framing was designed and sized for gravity loads only. The columns were sized to have maximum stress of 0.5 f_{ck} per the Turkish Seismic Design Code (MPWS, 2007), but under factored gravity loads excluding seismic loads. For beams, end moment capacity equal to the factored gravity loads was provided. However, recognizing that the beams would go under several cycles of reversal of the end moments in a strong seismic event, the same positive and negative bending capacity was provided at the beam ends to prevent premature hinging of the beams in a seismic event. Additional positive bending reinforcement was provided at the center of the beams to prevent formation of a hinge in the middle of the beams due to redistribution of forces.

PERFORMANCE EVALUATION – 475 YEAR EARTHQUAKE

An ETABS model of the structure, including the concrete shear walls and link beams all the way to the foundation level, as well as the columns and beam framing was constructed (Figure 4). The building core is a coupled wall system. The concrete core walls vary in thickness along the height. The core thickness is 110 cm below the 11th floor, 60 cm above the 22nd floor, and 85 cm in between. Link beams connect the core piers throughout the height of the structure. The link beams have rectangular cross sections with diagonal reinforcement. The width of the link beams matches the thickness of the core wall and the depth of the link beam is a function of the height of the opening and story height. Because of the high depth to length ratio for the beams, diagonal rebars were deemed more effective.

Spectral analysis was performed to calculate the seismic demands and to calculate the force demand and deflections. No reduction factor was applied to the response spectrum. Experimental data of low amplitude vibration of buildings of various construction and materials shows that a concrete structure of this height can easily have 2% damping as a baseline value (CTBUH, 2008). On the other hand, for a linear analysis based on assumption of highly nonlinear behaviour, 5% damping is the commonly used value. For the DBE-1 seismic level, because the response is allowed to reach the elastic limit and slightly exceed it, a damping value of 3% was deemed suitable.

Table 2 shows the modal period and mass of the first few modes of response. As seen below, for a structure of this height, the first modes in either major direction, while significant, do not represent more than about half the seismic mass. An appropriate modal analysis including at least 90% of the mass in all the major directions requires inclusion of several modes. The presence of higher modes which have lower periods and hence higher response spectrum accelerations results in considerable contribution to base shear.

Based on analysis results, it was verified that the structure and all its components met the IO performance targets. For the building, the strict maximum drift ratio limit of 0.5% was met (Figure 5). The core shear and overturning moments from the model were less than the capacity of the core. The moment and shear forces in the link beams and regular beams were checked against the expected member strengths times the m-factors and were found to be acceptable. Similar analysis was performed for shear and bending of columns. For column axial forces, the forces were checked

directly against the lower bound specified strength and were also found to be within limits. It is important to note that the columns and beams had to be checked to make sure that they met the IO requirements, although they were not considered part of the lateral system. The reason is that they have to move in a manner compatible to seismic deformations and have to be able to withstand the forces and deformations caused by that movement.

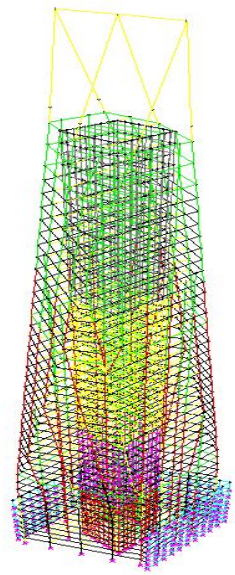

Figure 4- ETABS model for evaluation of seismic performance in linear range. The rooftop structure is only represented here for its mass contribution

Table 2- First few building modes from the ETABS model of the tower

Mode	Period (s)	UX (%)	UY (%)	RZ (%)	Sum UX (%)	Sum UY (%)	Sum RZ (%)
1	4.16	0.17	54.75	0.02	0.17	54.75	0.02
2	3.72	55.91	0.16	0	56.07	54.9	0.03
3	1.36	0.01	0	59.13	56.08	54.9	59.15
4	0.86	14.41	0.08	0.03	70.49	54.98	59.18
5	0.85	0.04	16.69	0.01	70.53	71.67	59.19
6	0.5	0.01	0.01	8.78	70.54	71.67	67.97
7	0.39	7.3	0	0	77.84	71.69	67.97
8	0.34	0	6.08	0.18	77.85	77.77	68.15
9	0.3	0.04	0.15	7.23	77.88	77.92	75.38

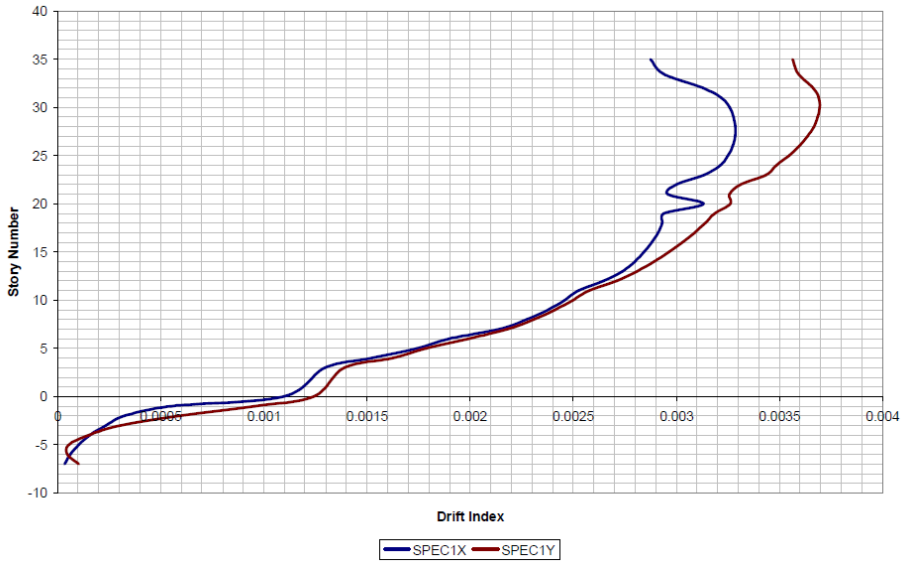

Figure 5- Story drifts under DBE-1 earthquake

PERFORMANCE EVALUATION – 2475 YEAR EARTHQUAKE

A nonlinear core-only model of the structure, as well as a core + frame model (Figure 6) were developed in Perform 3D (CSI, 2011). Both models had the structural mass of the entire building assigned to them and used expected material properties. The core-only model was intended for a direct understanding of the behavior of the principal lateral system, the core. The structural walls and coupling beams were designed as a special structural was system. The flexural response of the shear walls was modeled using fiber elements in Perform 3D. These elements consider the nonlinear axial stress-strain response of concrete and steel explicitly. The elements act elastically in weak bending and shear. Since the cracking of concrete and the resulting reduction in flexural stiffness was modeled explicitly using fiber elements, no further reduction was made to the flexural stiffness of the wall. To account for shear softening of the walls after cracking, the shear modulus was reduced by a factor of 4. However, the drift values were not very sensitive to this value. The beams were defined using standard frame elements, with concentrated plastic hinges at the ends. This was done using "FEMA beam" elements in Perform 3D, which are equivalent to the described model. The plastic hinge capacity of the beams was considered to be the same in positive and negative bending. The columns were defined using elastic frame elements and checked to verify that they did not exceed their capacities.

Since the tower was free to vibrate only above the B1 level and the ETABS model including the full basement showed limited effect on building periods, only this portion of the structure was modeled. The model included P-delta effects. Nonlinear time history analyses were performed using the earthquake records provided to us to verify that the structure met the Life Safety performance criteria. The only criterion that was not fully satisfied in the core-only model was the drift limit. Knowing that the framing would have some contribution to the stiffness, especially at upper floors, the core was designed to be close to meeting the drift requirements, such that it would meet them when the framing was added to the model. In the next step, the structural framing was added to the model to verify that all elements satisfied the Life Safety performance limits.

Because the nonlinear model explicitly captures energy dissipation through nonlinear behaviour of structural components, only the baseline damping related to other mechanism should be added directly to the model. As discussed previously, for a concrete structure of this height, 2% damping is appropriate. As such, Rayleigh damping with values ranging between 1.5-2% was assigned to the structure in the period range of interest.

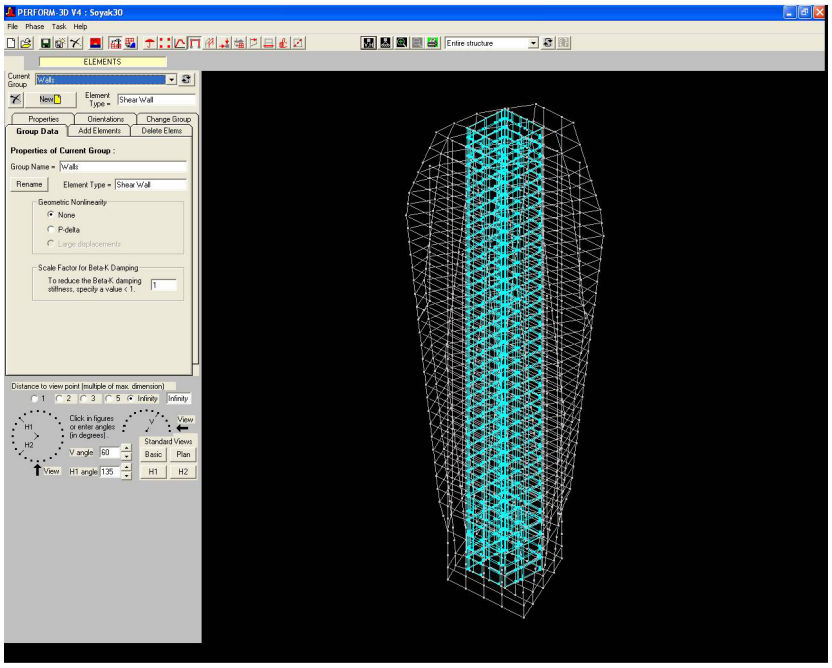

Figure 6- Nonlinear model of the structure in Perform 3D

The dynamic analyses were performed for the seven pairs of ground motions, alternatively applied in the X and Y direction, with two different gravity load combinations, resulting in 28 nonlinear dynamic analyses for each case. For any iteration of the design, running the full family of nonlinear analysis cases would take a few days. Once the results were obtained, they were post-processed within and outside Perform 3D. For deformation-controlled actions such as plastic hinging of beams or yield strain of rebars, mean demands were compared against allowable values for LS. For force-controlled actions such as axial forces on columns or shear demand for the core, mean demands were compared against strength based on lower bound specified structural properties. The sizes and strength of various components were updated until the Life Safety limit states were satisfied. This included limiting the drift to 1% (Figure 7). Figure 8 and Figure 9 show the story overturning moments and shear forces throughout the height as a result of the 28 nonlinear dynamic analyses. The mean demands were used for the design according to the ASCE 41 guidelines. A comparison of the maximum and mean forces shows the extent of variation in forces that is inherent to the characterization of seismic demands.

A very important observation from Figure 9 is the value of the base shear from nonlinear analysis. The mean base shear is about 17% of the weight, significantly higher than the base shear of 3% of weight that would be obtained from the Turkish seismic design code (MPWS, 2007). This is a shortcoming of standard forced-based design codes, not only the Turkish code, when they are applied to highrise structures. Whereas the base overturning moment in each direction is dominated by the first mode, the higher modes can significantly increase the shear forces throughout the height, including at the base. The nonlinear behaviour of the structure has limited effect on these higher modes. As a result, when actual shear forces are estimated using nonlinear analysis, the obtained values can be several times higher than the values implied by design codes. This phenomenon is not limited to this tower, but is applicable to other highrises as well. For the Crystal tower, the core was capable of carrying these large forces without changing the design. However, other highrise structures might not be as fortunate and might have significant underdesign in shear if they do not perform PBD. An engineer who understands this phenomenon can design the structure for a proper combination of strength and ductility for the shear forces. However, this phenomenon is not widely understood. Had the capacity of the core for the Crystal tower not been sufficient, PBD would have alerted the design team to the problem. This highlights an important benefit of PBD. It can help detect features that might be unknown, or unique to a special project, and that could have been missed in a standard design.

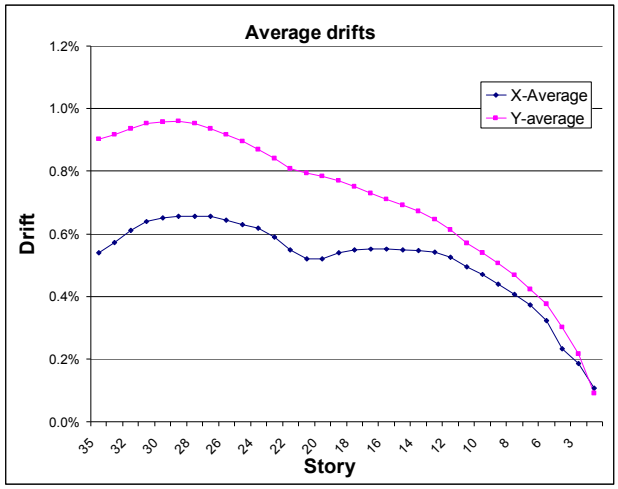

Figure 7- Mean peak story drift ratios from nonlinear analysis

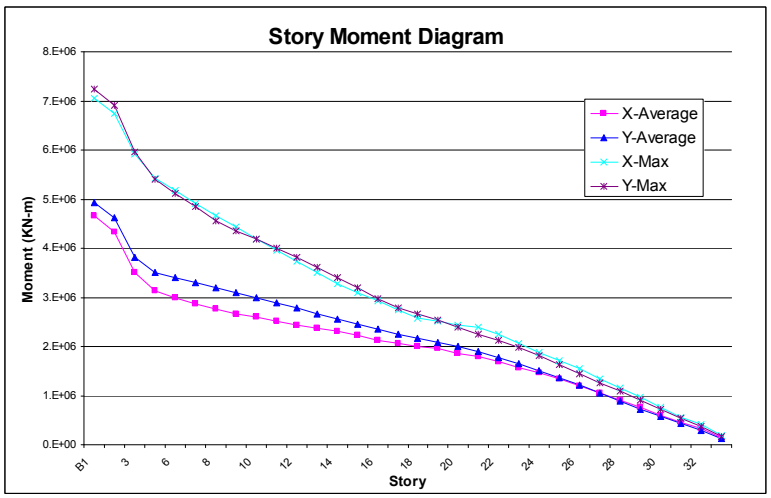

Figure 8- Story overturning moments from nonlinear analysis

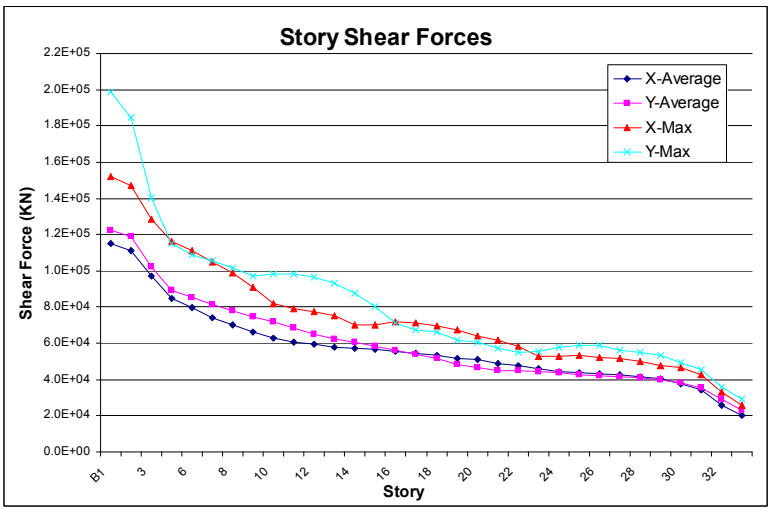

Figure 9- Story shear forces from nonlinear analysis

DESIGN OF CERTAIN CRITICAL ELEMENTS

Beyond the general check of members, there were several critical areas of the structure that required additional attention. One category of such critical area included several very short framing beams that were very close to columns or the core (Figure 3). Because of their large depth to span ratios, these beams were thought to attract significant force, but to lack the flexibility or strength to sustain these forces. Throughout the design, these critical beams were evaluated and where necessary, the framing was modified to eliminate these beams to ensure that they were not damaged during a seismic event.

The most critical elements of the structure after the concrete core were the floor diaphragms and especially the horizontal tie beams that carried to the core the large horizontal kick forces caused by changes in orientation of the columns. These forces were a function of the axial forces of the columns. The tie beam design was carried systematically, starting at calculating the appropriate axial forces for the columns. The column forces used to determine the tie forces at the column kinks were defined as $P_u = \alpha\ f_{ck}\ A_g$. To determine the α factors, the axial forces at the columns at each of these joints were considered under the worst nonlinear dynamic analysis case. In addition, some contribution was added to reflect the effect of the vertical component of the earthquake, although this effect is customarily not included in the design (ASCE, 2006). Since by design the tributary gravity weight on any column was less than $0.5\ f_{ck}\ A_g$ under the 1.4D + 1.6 L load combination, the actual seismic weight carried by it would be at least 30% less. On the other hand, the average vertical peak ground acceleration for the 2475 year event was 0.3g (Erdik, 2006). Since the vertical stiffness in structures is typically very large, the vertical component effect is estimated to be $0.3(1-0.3)\ 0.5\ f_{ck}\ A_g \approx 0.1\ f_{ck}\ A_g$ .To provide an extra margin of safety, an extra $0.1\ f_{ck}\ A_g$ was added to the force in the columns when calculating the kink forces, to ensure that these joints and their load paths to the core would not fail. In summary, $0.2\ f_{ck}\ A_g$ was added to the maximum axial force observed in the columns during any of the dynamic analysis cases to provide an upper bound estimate of the force that had to be carried at the tie beams to ensure the stability of the load path. This approach was appropriate given the importance of the tie beams to structural stability. This is similar to the design philosophy for critical components of highly ductile lateral force systems, where it is ensured that those critical elements can sustain loads above and beyond what other elements will deliver to them and hence, will stay intact during the earthquake. The maximum value of α for any column at a critical joint was 0.8, with a value of 0.6-0.65 being most common. Once the axial forces were calculated, calculation of the horizontal forces was straightforward based on the node geometry.

For the transfer of the force to the concrete core, there were three general load paths, with variations between floors. Figure 10 shows the load path at 3rd floor, the most complicated floor, where large kick forces were to be transferred from the exterior framing, the intermediate framing, and the center framing. The proper design included increasing slab thickness to 30 cm and 60 cm in different areas of the floor, insertion of additional rebars in visible tie beams or tie beams embedded in the slab, provision of ties for compression struts, and shear friction reinforcement to transfer forces form slabs into the core.

There were several other critical parts of the design that are not discussed here because of limited space. The layout and congestion of rebars in many areas had to be modelled and studied to ensure constructability. The transfer of the shear forces from the concrete core back into the foundation walls, adjacent soil, and foundation required careful methodical design. Also, the geometry of tie beams was such that the columns met at the top of the slab. Therefore, the large tie forces were very eccentric to the tie beams. Several measures were taken in design to ensure that the tie beams can carry the large eccentric forces adequately, and to ensure that no matter what, the tie beam integrity and the load transfer would not be affected.

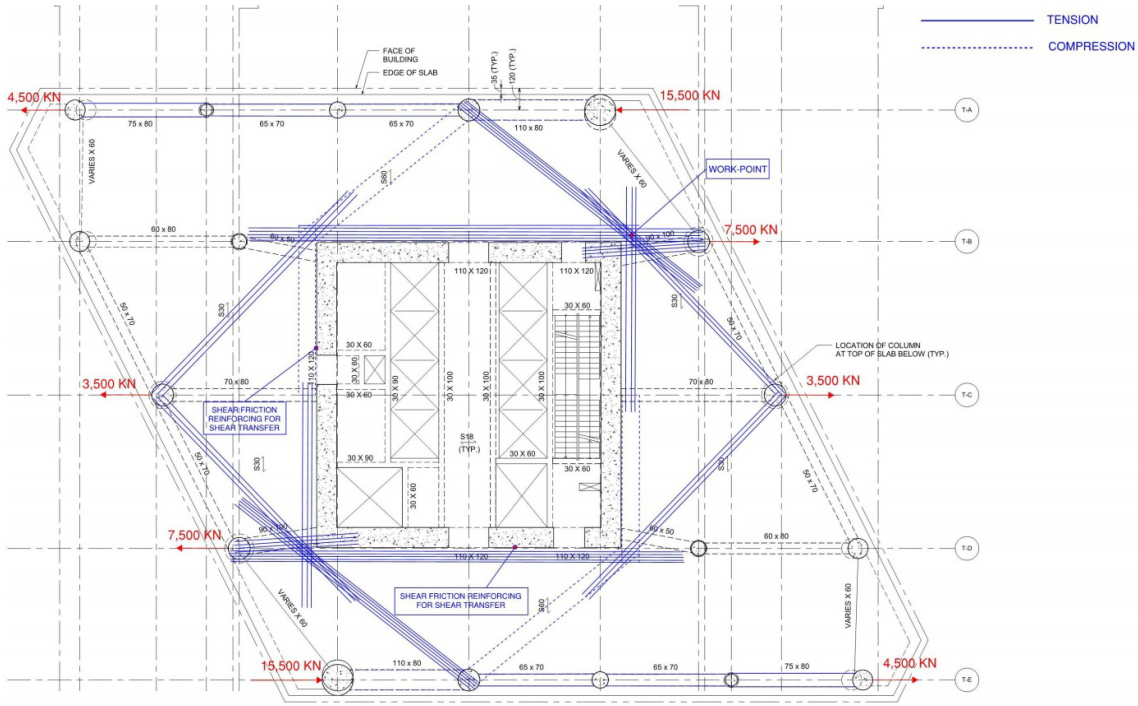

Figure 10- Load path within the 3rd floor to transfer horizontal kick forces into the concrete core

CONCLUSIONS

Several factors pushed the design for Crystal Tower towards the state-of-the-art performance based design approach. These factors included the higher expectations of the seismic performance of the structure by the owner, challenges of designing critical points in the structure reliably and safely using standard code-based approach, and the desire to remove unnecessary moment frames from the design and relying on the more efficient concrete core as the lateral system.

Performance based design allowed for achieving the above goals and more. The structure was successfully designed for Immediate Occupancy in a 475 year event and Life Safety in a 2475 year event. By modelling the realistic behaviour of the structure, including nonlinearity, and assigning stiffness and strength only where it helped the building performance, we were able to achieve a more reliable as well as economical design. The elimination of unnecessary moment frames and design of framing for only gravity loads allowed for a reduction in concrete volume of 45% for beams and 20% for columns, a significant cost saving for the client without any adverse effect on structural performance.

The reduction of the size of framing and elimination of many interior frames also had additional benefits to the functionality and aesthetics of the building. The elimination of some interior framing allowed for a better design of the buildings mechanical and electrical systems, and an increase in useable floor heights. Elimination of unnecessary perimeter moment frames also allowed for a 12 m tall open lobby space, a tremendous value to the client.

In terms of safety, PBD allowed for a demonstrably safe design of critical elements of the load path, which would not have been reasonably possible using standard design. Furthermore, the analysis showed that certain global effects such as seismic shear of highrise buildings are consistently and seriously underestimated by the standard design approach. Whereas this did not pose a problem for the Crystal tower, it might well be a source of unconservative design for other highrise designs with standard force-based approach.

In summary, performance based design was the right choice for this project, resulting in a more economical, more reliable, more functional building with increased beauty for the building and satisfaction for the client.

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