PERFORMANCE BASED SEISMIC AND WIND ENGINEERING FOR 60 STORY TWIN TOWERS IN MANILA

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ABSTRACT :

This paper describes the structural design of two similar 60 storey towers in Manila using performance based procedures for seismic and wind actions. High-rise buildings designed by performance based methods not only perform better than conventionally designed ones, but are also less expensive to construct. The buildings incorporate the Arup Damped Outrigger System, and the savings realized by this are discussed.

KEYWORDS: High-rise building, performance based design, viscous dampers, wind, seismic

1. INTRODUCTION

St Francis Shangri-La Place, Manila, is a development of two similar 210m tall residential buildings approximately 38m square in plan located in a region of typhoon winds and in UBC-97 seismic Zone 4. Each building has a reinforced concrete core with an irregular arrangement of perimeter columns and walls, and incorporates a two storey deep outrigger system at approximately half the overall height. Each outrigger is connected to the adjacent perimeter columns by means of vertically acting fluid-viscous dampers.

The structural design of high-rise buildings is often governed by dynamic performance in winds, and in regions of high seismicity, by seismic performance. Conventional practice is to stiffen a building in order to reduce the dynamic response under wind loading. However, this has the effect of increasing the seismic forces the building must be designed for. By adding a robust supplementary damping system to the structure instead of stiffening, both wind responses and seismic forces are reduced – which leads to construction cost savings.

Conventional US Code seismic design procedures and their many derivatives used around the world (such as the NSCP - National Structural Code of the Philippines) were never intended for very tall buildings and are unsuitable for buildings of the height of these towers. This paper discusses the use of performance based seismic design for this project and illustrates the reasons why conventional code procedures should not be used.

2. STRUCTURAL FORM OF TWIN TOWERS

2.1 Structural system

The 60-story twin towers are illustrated in figure 1. Each building contains a central reinforced concrete core in two halves connected in one direction with diagonally-reinforced coupling beams. The thickness of the core walls increases towards the base to accommodate the increase in gravity load and seismic shear force. A perimeter moment frame provides both additional lateral resistance and gravity support for the concrete slab-on-beam floor. Eight double-story deep outrigger walls are attached to the core at the half-height of each building. Two vertically acting fluid-viscous dampers connect the end of each of the outrigger walls to the adjacent perimeter column as shown in figures 2, 3 and 4; there are a total of 16 damper elements per building.

2.2 Structural system classification

The building would be classified as a bearing wall system per the UBC-97 or the NSCP since the core walls support a significant proportion of the gravity load. The height limit in the UBC for this structural system is 160 feet (49m), just 25% of the height of these towers. The building cannot be described as a "dual system" since there is not an essentially complete gravity space frame independent of the main laterally resisting walls. Hence, this building falls outside code permitted limits, as would almost any building of this height.

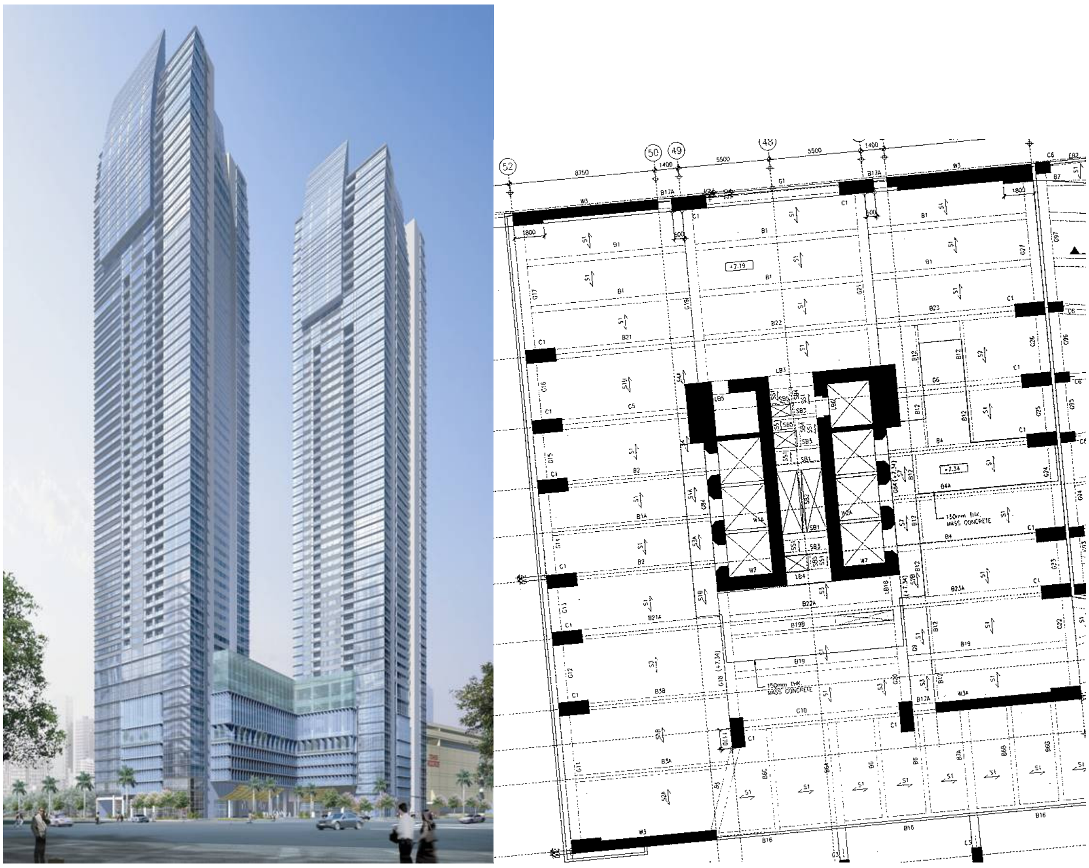

Figure 1 Architectural rendering of towers and structural plan of tower 2

3. DAMPING SYSTEM

Supplementary damping can be beneficial in reducing the wind-induced responses of high-rise buildings and tuned inertial devices (tuned mass, tuned liquid and active mass dampers) have been used previously many times. However, their dependence on tuning being maintained, and the lack of redundancy of many such systems means they have been relied upon only to reduce motion for occupant comfort. For the St Francis Towers damping has been introduced in the form of the Arup Damped Outrigger System (patents pending) using a highly redundant arrangement of multiple fluid-viscous dampers. This system is non-tuned, and its performance is insensitive to changes in the properties of the building or the dampers themselves. This robustness enables the significant benefit of reduced wind design forces to be realized (together with benefit in seismic response control). This system has the additional advantages of much lower space requirement and mass compared to tuned inertial devices, and can economically develop higher levels of damping.

3.1 How the Arup damping system works

Figure 2 illustrates the principle of the Arup Damped Outrigger. As a building undergoes dynamic sway motion, there is relative vertical motion between the ends of stiff outrigger elements cantilevering from the core and the perimeter columns. If dampers are inserted across this discontinuity, energy is dissipated during the cyclic motion resulting in increased overall damping of the building. Further information is given by Smith and Willford (2007).

3.2 Layout of dampers

The location of the dampers is illustrated in Figures 3 and 4. Two damper components are provided per wall instead of one since this is cheaper to supply, easier to install and provides greater redundancy

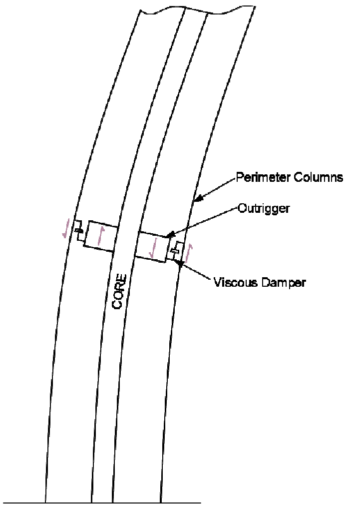

Figure 2 - Arup Damped Outrigger (patents pending)

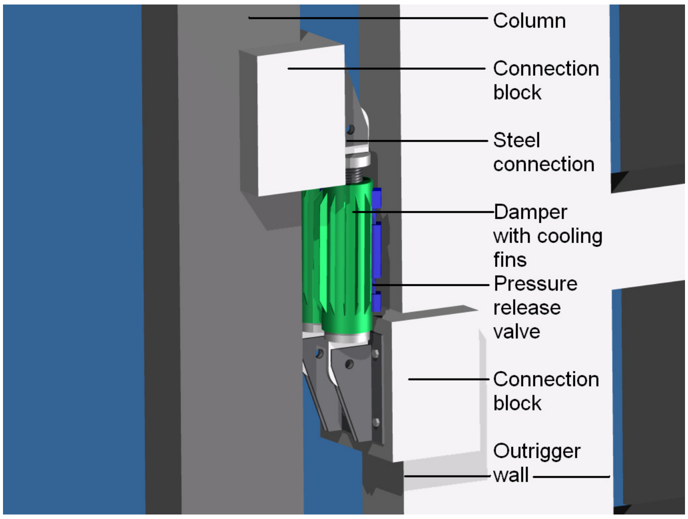

Figure 3 - Close up of damper connection detail

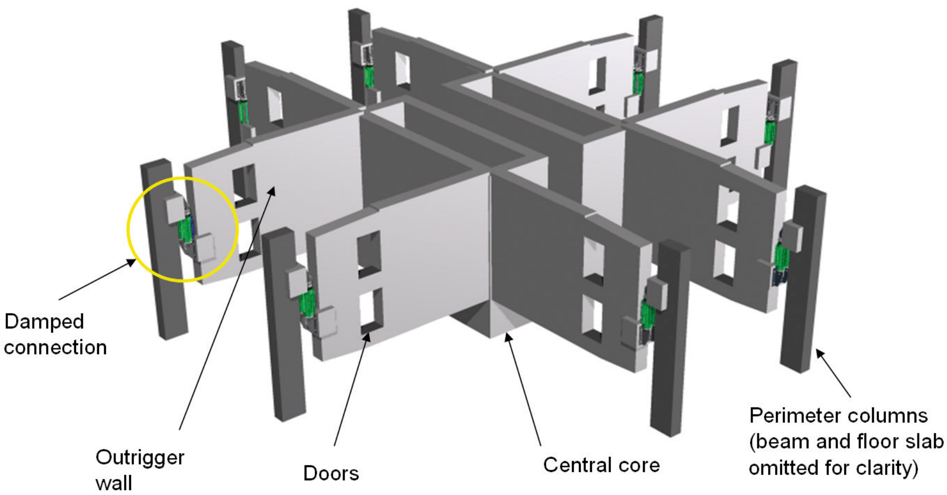

Figure 4 General arrangement of outrigger levels

3.3 Behavior of dampers

The dampers adopted for the St. Francis project have design non-linear characteristics as described below:

$F = C_1 v^2$ where F is the force developed, v is the velocity across the damper, and C_1 is a constant.

At an upper threshold velocity, the behavior changes to:

$F = F_1 + C_2\, v^{0.1}$ where C_2 is a second constant.

This change in characteristic prevents overload under extreme seismic demands and is implemented by the use of a pressure release valve. The threshold force F_1 is the maximum force seen in the dampers during wind storms. At low levels of building response the mobilized damper force is very low; this reduces the working pressure within the damper, increases the lifetime of the moving components and reduces susceptibility to fatigue. The damping system provides higher damping levels when the wind approaches the design speed, achieving between 5% to 11% of critical for the two buildings and two principal directions in the 100 year wind.

3.4 Damper strokes and forces

The peak stroke during the design wind storm is predicted to be approximately 50mm, and 200mm under the 2475 year return period earthquake. The peak force is 2.2MN per damper, with two dampers per outrigger.

3.5 Testing and Installation of dampers

3.5.1 Testing

The dampers were tested by the manufacturer, FIP Industriale, prior to shipment. This is described in a separate conference paper [Infanti, Robinson, Smith, (2008)].

3.5.2 Installation

All the dampers were installed in early 2008. Installation is relatively straightforward as shown in figure 5, requiring no specialist tools or commissioning process, other than heavy lifting.

Figure 5 Installation of Dampers

4. WIND INDUCED RESPONSE

4.1 Wind Effects

The inertia forces associated with the resonant dynamic response of a tall building in wind usually results in the lateral design forces for the structure being substantially higher than the applied aerodynamic forces. It is also common for the predicted wind-induced sway motion of a tall building to exceed human comfort criteria, and this may require the dynamic response to be reduced. The dynamic response of a tall building in a given wind environment is governed by a number of factors including shape, stiffness, mass and damping. Whilst the effect of shape can be assessed by wind tunnel testing and the mass and stiffness can be predicted with reasonable accuracy by the structural designer, the intrinsic damping of tall buildings is low, and the limited measurements available show significant variability (Smith and Willford, 2008).

By adding an engineered supplementary damping system to a building it is possible to remove dependence on the low and uncertain intrinsic damping. This improves the reliability of dynamic response predictions and, by supplying higher levels of damping, substantially reduces the dynamic responses (and the actual lateral forces experienced); the occupant comfort performance is also improved. By incorporating a damping system of adequate redundancy, reliability and robustness, reductions can be made to the structural sizes otherwise required, resulting in cost and construction time savings, and increasing the net floor area. This was the approach adopted on the St Francis Shangri-la Towers.

4.2 Prediction of wind-induced response

For scheme design the likely dynamic response was calculated using the Detailed Method of the National Building Code of Canada (NBCC) for both along-wind and across-wind directions. For occupant comfort 10 year and 1 year return periods were considered, and the 100 year return code wind speed was taken for strength design. Wind tunnel testing, using the high-frequency force balance technique, was later used to refine the static and dynamic wind forces. A site-specific directional wind climate study was also performed, which made a significant difference to the final assessment of wind induced response.

Considering one direction only, the ultimate design overturning moments were as follows.

Method	**Windspeed**	**Overturning moment GNm** **(1% intrinsic damping)**	**Overturning moment GNm** **(7.5% damping)**
NSCP or ASCE 7	Code	6.0	-
NBCC	Code	15.2 (crosswind) 6.7 (along wind)	5.0
Wind tunnel	Code	7.4	4.5
Wind tunnel	Climate study	-	3.7

Table 1 Variation of wind overturning moment with method, wind speed and damping

A number of observations can be made on the different methods used, and by comparing values in this table.

- ASCE 7 and the NSCP do not provide predictions for cross wind response which may be critical for tall slender buildings such as these towers, and the overturning moment predicted by wind tunnel test is 23% higher than these codes predict
- The NBCC cross-wind prediction is conservative for these buildings compared to the result from wind tunnel testing
- The use of fluid-viscous dampers reduces the design overturning moment by 40%
- The site specific wind climate study reduces the design overturning moment by a further 18%.

4.3 Lateral accelerations

The lateral accelerations in Table 2 were predicted for the 10 year wind, based upon the wind tunnel tests and the wind climate study. From the results, it is clear that the provision of the supplemental damping system changes a building with unsatisfactory performance for residential occupancy to one with excellent performance.

Damping		**Peak acceleration (milli-g)**
Intrinsic damping	1.0% of critical	25.6
Total damping	7.5% of critical	9.4
Suggested limit for residential building (7 seconds period)		15

Table 2 Variation of lateral acceleration with damping

5. SEISMIC RESPONSE

5.1 Background

Current seismic design provisions in building codes, such the International Building Code (IBC 2006), the NSCP and Eurocode 8 (2004), were not developed with tall and ultra-tall buildings in mind since such buildings comprise a very small percentage of the overall building inventory. The focus of these codes is low-to-medium-rise buildings. As a consequence, the codes do not consider lateral-force-resisting systems that are widely used and economical for tall and ultra-tall buildings. Equally, the prescriptive procedures in these codes lack provisions necessary to ensure acceptable behavior of tall buildings when shaken by strong earthquake ground motions. In order to design safe and economical tall buildings in seismic regions, it is necessary to consider their expected response using first principles engineering mechanics.

Performance-based seismic design is permitted by building codes as an alternative to the prescriptive procedures. For instance, Clause 1629.10.1 of the UBC 97 states 'Alternative lateral-force procedure using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions'. However, next to no specific guidance is presented as to how this should be carried out. The CTBUH (2008) has recently published guidance to assist.

5.2 Changes to structure made possible by performance based design

Once the engineer is free of the prescriptive rules found in conventional building design codes, it is possible to design structural systems which are appropriate for tall buildings and which will perform well in earthquakes. Relative to a building code design, the changes made to the St Francis Towers include:

1. The moment frames were not designed to resist 25% of the base shear, which would have been required if the building were designed as a code 'dual system'.
2. The building was made more flexible to reduce the seismic base shear.
3. The bending strength of the tower was reduced to that required for factored wind loading.
4. The core walls were made thicker at the base of the building to reduce gravity stress and permit flexural ductility
5. Elsewhere, where there is no inelastic demand, walls did not need to be sized to be working at less than $0.35Af_c$', resulting in reduced thickness and reduced confinement reinforcement.
6. The shear strength of the walls had to be increased
7. The beneficial effect of the viscous dampers could be represented in the seismic analysis

Whilst most of these changes led to reduced member sizes and construction cost, the increases noted at points 4 and 6 were essential to ensure a safe design; the code did not provide adequate checks.

5.3 Design objectives

The key objective set for these buildings was 'life safety' performance for the MCE earthquake, a higher standard than that implicit for code-designed buildings. The MCE was determined by site specific hazard assessment based on a 2475 year return period.

5.4 Analysis procedures

Non-linear response history analysis was chosen as the only appropriate method to determine the response of the buildings for the MCE ground motions. Appropriate input ground motions were spectrally matched in the time domain to the target MCE response spectrum using RSPMatch. The software used for non-linear structural analysis was LS-DYNA, and element strengths were taken at 'expected value' level. Six separate bi-directional time history runs were performed with the worst case envelope values considered in design.

5.5 Analysis results

5.5.1 Global forces for one direction

Table 3 compares the global seismic overturning moments and shear forces obtained by various methods with the corresponding design wind effects.

Method	**Assumption**	**Overturning moment (MNm)**	**Base shear (MN)**
NSCP – code seismic	Designed as bearing wall system (R=4.5)	10400	130
NSCP – code seismic	Designed as dual system (R=8.5)	5505	68
Wind (from wind tunnel)	Factored design wind force	3700	16
Non-linear MCE seismic	Expected strengths	4690	99
MCE Elastic response	No ductility R=1	12850	160

Table 3 Global seismic and wind forces

It can be seen that the overturning moment arising from the performance based design is significantly less than the bearing wall moment in the "code" design. The margin between the factored wind and seismic overturning moment represents the over-strength of the building. It can also be seen that had the building been designed as a dual system, then the design base shear would have been significantly underestimated. Being a force-controlled action, the shear must be designed for.

5.5.2 Element deformations

For elements (and deformation modes) which behave in a ductile manner the predicted plastic hinge rotations (the ductility demand) are compared with allowable limits (ductility capacity). Guidelines for maximum permissible hinge rotations were taken from FEMA 356 (now superseded by ASCE 41, 2006), and first principles calculations were used for situations outside the scope of FEMA tables. Sample values are given in Table 4.

Element	**Predicted hinge rotation (radians)**	**Life Safety Limit from FEMA 356**
Base of wall	0.0024	0.006
Floor beam	0.018	0.020
Base of column	0.008	0.012

Table 4 – Plastic hinge rotation in typical elements

5.5.3 Element forces

Actions in non-ductile modes such as compression and shear in reinforced concrete are known as 'force-controlled' actions. In this case, the expected forces are taken from the analysis and the elements designed for these values (assuming code-level strength) to ensure they are not subject to brittle failure.

5.6 Peer review

This project was subjected to a high level peer review looking at overall design strategy, methodology, assumptions, key analysis results and design details. This role was performed by Prof. Andrew Whittaker of the University at Buffalo, NY, USA.

6. ECONOMIC BENEFITS

By the use of performance based seismic design, wind tunnel testing and a viscous damping system, it was possible to reduce very significantly the design forces on these tall buildings at the same time as providing a superior standard of wind motion serviceability and seismic performance and safety. In this project the new design has 30% less concrete, and reinforcement densities are reduced from 300 kg/m^3 to 200 kg/m^3 compared to a 'code' design. Allowing for the cost of the dampers, the net construction saving for the project (at Philippines prices) was in excess of $5M US.

7. CONCLUSIONS

Studies performed for the design of these buildings demonstrate factors relevant to many high-rise buildings. Firstly, the inclusion of a robust non-tuned supplementary damping system can substantially reduce wind load effects, permitting more economical structural design and reducing the risks associated with uncertain intrinsic damping. Secondly, performance based seismic design is essential for such buildings. Blindly following 'code' procedures restricts the design to unsuitable and uneconomic structural forms that will not perform well in strong earthquakes because the shear demand on the building may be underestimated and it is unlikely that the required flexural ductility at the base of the core would be achieved. If the prescriptive code rule regarding proportion of shear to be resisted by moment frames is followed, the structural cost can increase dramatically with minimal effect on performance. Thus, paradoxically, 'code-designed' high-rise buildings can be both more expensive and less safe than those designed by performance-based techniques.

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