Robust Modelling and Validation Analysis on the Raffles City Chongqing Project

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Abstract

Since 2012, Arup has been providing structural engineering services for all design stages of the development of the Raffles City Chongqing (RCCQ). This truly iconic mega-complex has been designed by the internationally acclaimed architect Moshe Safdie for the developers CapitalLand and Singbridge Holdings.

Various structural schemes for the 300m-long sky deck atop the four 250m-high interior towers have been assessed using LS-DYNA [1]. Extensive nonlinear time-history analyses have been performed to simulate the behavior of several conservatory articulations and isolation solutions under extreme seismic conditions.

Arup Shanghai has also been investigating innovative fuse/concrete outrigger solutions to meet the wind/seismic demands on the 350+m North Towers (T3N and T4N). A hybrid steel diagonal and concrete wall outrigger system (Hybrid OT wall) proved particularly promising. Compared with traditional steel designs, a Hybrid OT wall would simplify the design of the wall-to-mega-column connection while being significantly cheaper in cost.

The development of these elements required extensive Finite Element analyses and physical testing. By deploying advanced LS-DYNA capabilities, structurally reliable and cost-efficient options have been identified and validated.

This paper presents the validation process, analysis results and the design solutions that could achieve the architecturally ambitious, safe and sustainable design while providing significant cost savings.

***Keywords**: LS-DYNA performance-based design, seismic isolation, friction pendulum bearings, viscous dampers, fuse and outrigger wall system, *MAT_WINFRITH_CONCRETE*

Figure 1: Raffles City Chongqing – Courtesy of Moshe Safdie Architects [2]

A new landmark and specific engineering challenges

The super-scale development of Raffles City in Chongqing, for which Arup has been appointed to provide structural engineering services for all design stages, will become a new landmark for the region. Located at the heart of Chongqing, at the junction between the Yangtze and Jialing rivers, the site is charged with historic and symbolic significance.
The super-scale development design by internationally acclaimed architect Moshe Safdie is inspired by images of great Chinese sailing vessels on the river. It will pay tribute to Chongqing's noble past as a trading centre and also serves as a symbol of the city thriving present and promising future.
The complex will comprise a shopping mall and eight towers for residential, office, serviced apartment and hotel use yielding a total of GFA exceeding 1.1 million square meters. The development will also serve as a major transportation hub integrating bus, ferry terminals and subway station. Six slender towers will sit atop of a five-storey retail podium; 'gently arching towards the water, they will form the apex to the city peninsula – like the great masts of a ship, with its sails pulling the city forward' [2].

A 280-meter long, glass clad conservatory that bridges the four interior towers at the 60-storey level, 250m-high, providing various amenities, green space and 360-degree views of both rivers, is a key architectural element. And the design of the structure, in such a seismically active region, presents particular engineering challenges.
From the Concept Design stage, the complexity of the bridge coupling options and the nonlinearities of the articulation mechanisms required thorough assessments using LS-DYNA. Detailed three-dimensional models of the four supporting towers and the conservatory were developed to capture the dynamic interactions under severe seismic excitation and validate the articulation solution and the seismic resilience of the structure.

Arup has also been investigating innovative fuse/concrete outrigger solutions to meet the wind/seismic demands on the 350m-tall North Towers. A hybrid steel diagonal and concrete wall outrigger system (Hybrid OT wall) proved particularly promising. Compared to traditional steel designs, a Hybrid OT wall would simplifies the design of the connection wall-to-mega column and be significantly cheaper.

However, the development of these elements required extensive FE analyses prior to any physical test validation performed by the laboratory of the China Academy of Building Research (CABR).
This paper describes the robust modelling techniques that have been developed for analyzing these novel elements.

Isolation Scheme of the Conservatory Bridge

In the Concept and Detailed Design stages of the project, full seismic isolation of the conservatory bridge proved to offer compelling benefits. The main advantages and the preliminary investigation analysis work are summarized in *'Isolation Scheme Assessment on the Raffles City Chongqing using LS-DYNA'* [3]. In particular, a floating conservatory design allows:

- A reduction of the seismic forces in the bridge truss and consequent saving in steel tonnage and construction cost

- A reduction in shear forces at the Towers/Conservatory interfaces resulting in a simplified and cost-effective connection design
- A simplification of the construction sequence
- A continuous conservatory cladding as required by the Architect

These potential savings and benefits are partly offset by the higher bearing/damper cost and MEP complications in the articulation zone between towers and conservatory.
Figure 2 below illustrates conservatory/tower interface options of different Arup-design iconic projects.

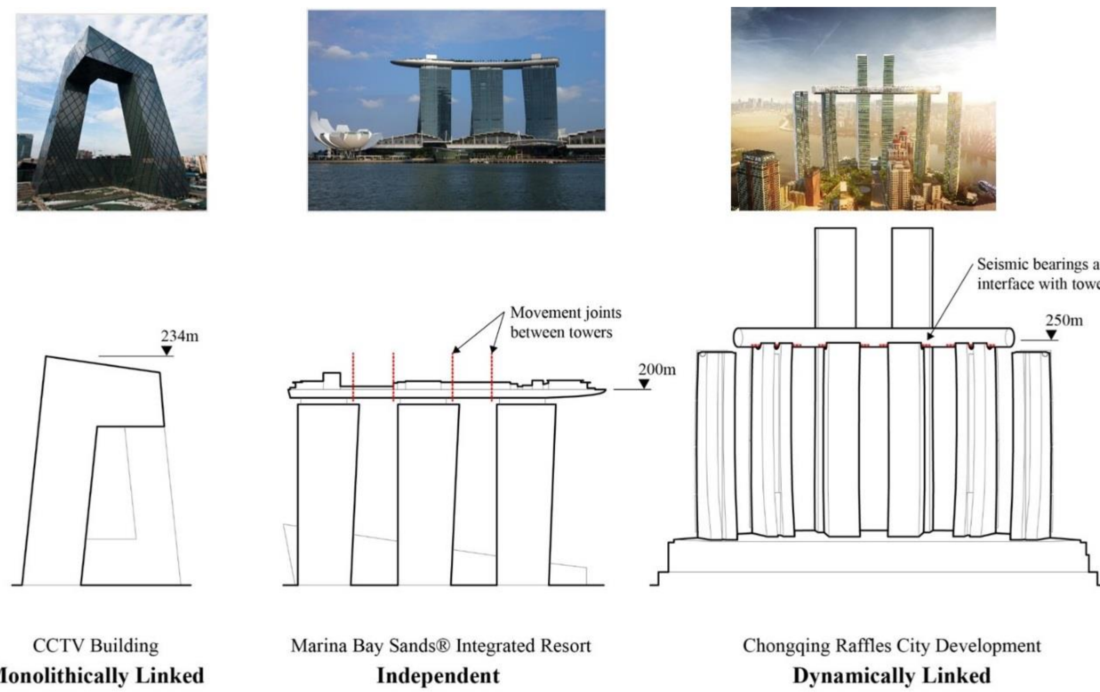

Figure 2: Conservatory/towers interface design

The optimum scheme selection required a thorough cost/benefit analysis at each stage of the design.
As the tower and bridge design evolved, various rounds of optimization of the isolation options were performed. The following sections illustrate the design process of the final bearing/damper scheme and the validation analyses.

Analysis model – South Towers and Conservatory Bridge

From the initial Concept stage, different specifications and types of bearings were considered. Cost/benefit analyses were performed for Lead Rubber Bearings (LRB) and Friction Pendulum Bearings (FPB). Options combining FPBs and viscous dampers were also envisaged. The main contribution of the viscous dampers, apart from increasing the damping, is to reduce the displacements at the conservatory/tower interface in both longitudinal and transverse directions.

Analysis process

Due to the highly nonlinear behavior of the Friction Pendulum bearings and the possible phase differences between the tower displacements, response spectrum analysis methods are not appropriate.

Instead, the seismic response of the four supporting towers and the conservatory bridge structure to the Level 3 Maximum Considered Earthquake (MCE) was simulated in LS-DYNA (971 R6.1.0) by nonlinear time-history analysis method. The *MAT_SEISMIC_ISOLATOR material is also particularly well suited to model the nonlinear force-displacement hysteretic behavior of the FPB/LRB. For a detailed description of this LS-DYNA capability, cf. [1].

Ground motion excitation

The details of the project acceleration response spectrum and prescribed ground motions, corresponding to the Level 3 MCE seismic event were given in [3]. The same seven sets (5 natural and 2 artificial records) of spectrum-compatible ground motion time-histories were applied for nonlinear transient analysis in the Detailed Design and Validation stages.

Towers and conservatory models

Early assessment models included elastic representations of the towers and the conservatory (cf. [3]). But for the isolation scheme Detailed Design and Validation stages, fully nonlinear models were developed to properly capture member damage, hysteretic energy dissipation and structural period elongation effects.

The design of the structure was constantly updated as the project advanced. Details of the conservatory/tower interface were introduced in the analysis models as design choices were being finalized. For instance, the location and size of the elevator shafts, fire escapes and the detailed structure of the pedestrian link bridge to the North Towers imposed additional constraints on the articulation scheme.

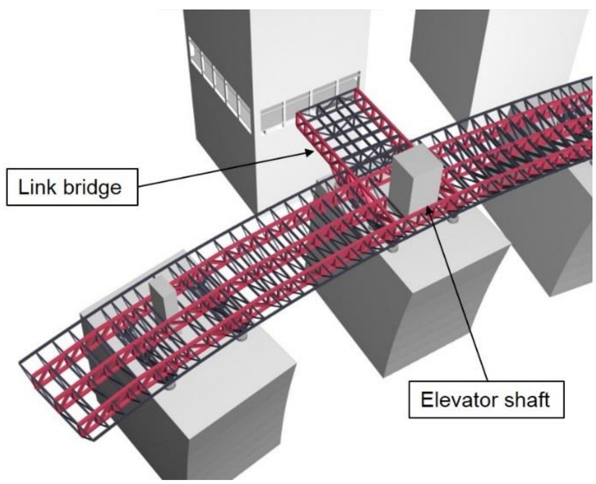

Figure 3: Conservatory link bridge and elevator shaft details

Assessment of seismic demands and damage of structural members

The same comprehensive model was used to assess the seismic performance of the conservatory and of the supporting towers. The complete analysis model is illustrated in Figure 4:

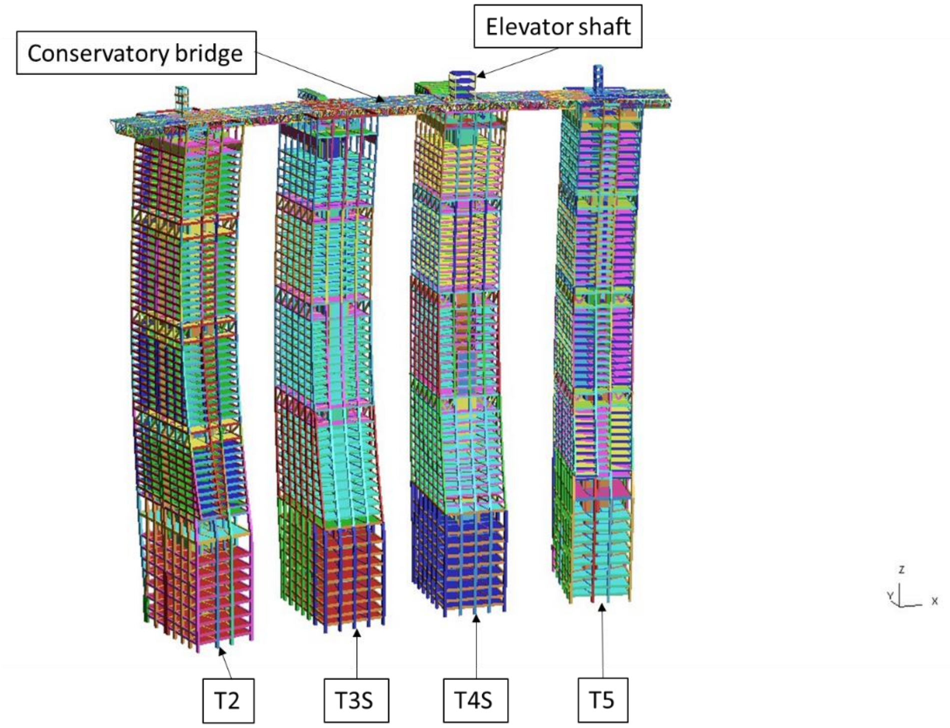

Figure 4: Complete LS-DYNA analysis model

Interface elements, FPB, viscous dampers were modelled explicitly, see Figure 5 below. Detailed design and locations of the connections were updated as the structural envelope was progressively frozen.

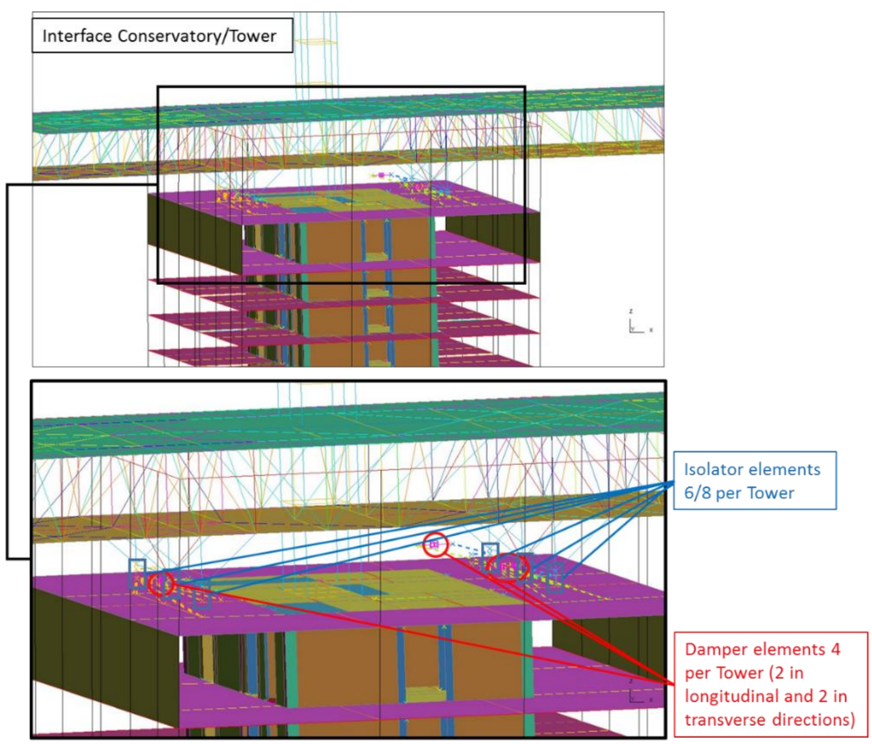

Figure 5: Details of conservatory/tower interface – Final Scheme

Maximum seismic force and displacement demands during the entire seismic events were compared with the structure's capacities on a component-by-component basis. Figure 6 below compares the critical component damage with FEMA 356/ASCE 41-06 criteria [4] under the MCE seismic event.

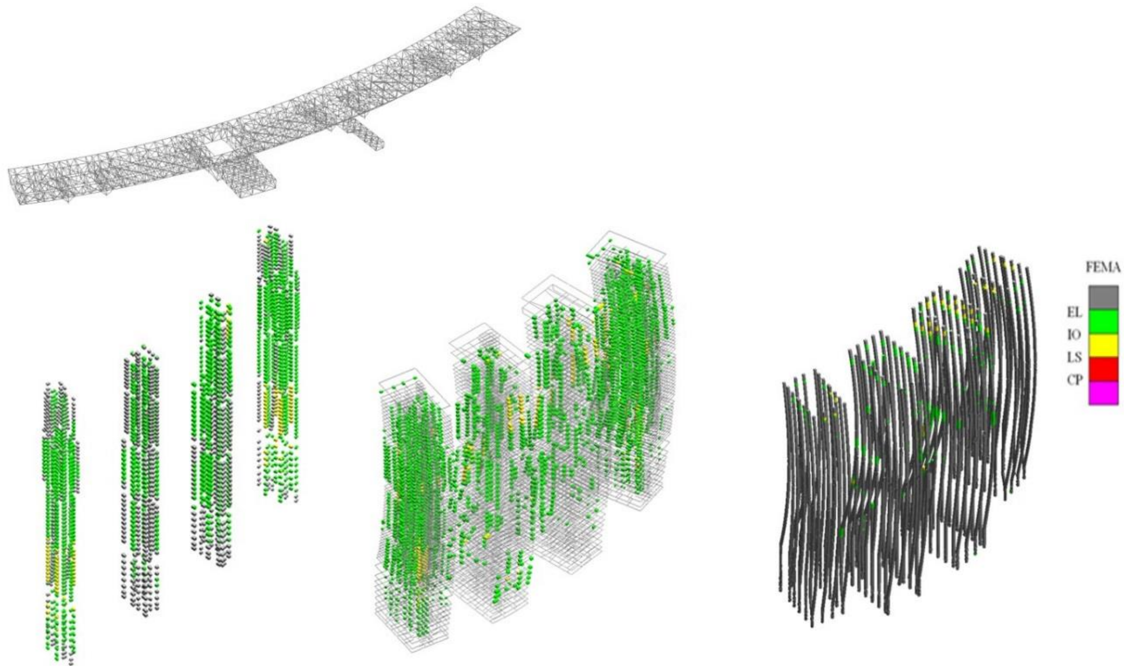

Figure 6: Critical member damage – FEMA criteria – Typical MCE event

For all Level 3 ground motions considered, the damage levels in the critical structural tower components, as per FEMA 356/ASCE 41-06 criteria, remain within Immediate Occupancy limits (IO) and are deemed acceptable.

Realistic modeling parameters and provisional cost estimates for bearings and dampers were obtained from established suppliers. These quantities were also adjusted as the dynamic properties of the individual towers and conservatory changed through the various design iterations.

As the building design was refined, a fine balance between bearing displacements/damper strokes, interface and conservatory demands and structural costs was sought to achieve the most economical compromise. A combination of FPB with 5% dynamic friction and viscous dampers rapidly emerged as the most cost effective and feasible solution.

The final articulation scheme is illustrated in Figure 7 below. This layout takes into account the latest geometry of the conservatory support truss and the location of electrical and mechanical plants at the tower interface. It also aims at facilitating maintenance access and reducing congestion while maintaining isolation efficiency.

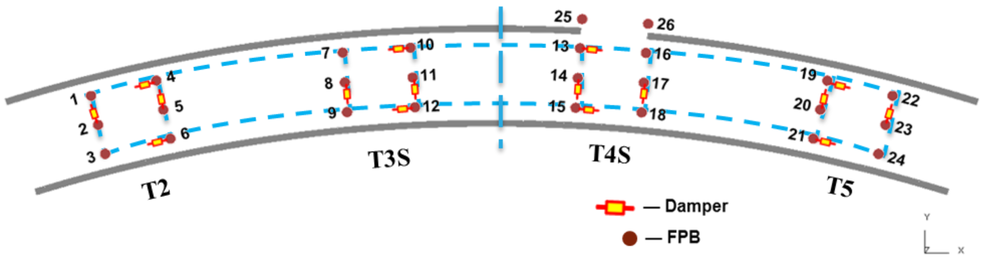

Figure 7: Final articulation scheme

Table 1 below summarizes the properties of the optimized FPB design:

	Design parameter	**Value – optimized scheme**
FPB	Configuration	6 no. at Towers T2, T3S and T5 8 no. at Tower T4S
	Maximum displacements	400mm at Towers T2, T3S and T5 450mm at Tower T4S
	Curvature radius	4.5m
	Dynamic friction coefficient	5%
	Average service load	15MN
	ULS specified load	28MN at Tower T2 33MN at Towers T3s and T4s 26MN at Tower T5
	Internal hinge rotation	+/-0.01rad

Table 1: Final articulation scheme – FPB characteristics

The viscous damper parameters applied during the Detailed Design and Validation stages are given in Table 2:

	Design parameter	**Value – optimized scheme**
Seismic dampers	Constitutive law	$F = CV^{\alpha}$ F: Force [N] V: Velocity [m/s] α : Velocity exponent C: Constant $[Ns/m]^{-\alpha}$
	Law parameters	$\alpha = 0.3$ $C = 5e6MN/(m/s)^{-0.3}$ tuned to obtain 5000kN at V=1m/s
	Configuration	4 no. per Tower (2 in longitudinal, 2 in transverse directions)
	Maximum stroke	400mm at Towers T2, T3S and T5 450mm at Tower T4S
	Integrated fuse function	Fuse release force 2MN at approx. 1.5mm/s

Table 2: Final articulation scheme – Viscous damper characteristics

Analysis results – Conservatory isolation scheme

A summary of the performance of the isolated scheme is provided in this section. The optimized isolated scheme is compared with the fixed conservatory option. The estimated tonnage saving for the conservatory structure is also reported.

Design options	Description
Baseline – Option 1	Conservatory rigidly fixed at all support towers
Optimised isolated scheme – Option 2	Conservatory isolated at all 4 towers Combination of FPB and viscous dampers Optimised mechanical properties

Table 3: Analysis cases

Predicted forces

Predicted maximum shear forces at the conservatory/tower interface both peak values and averages over the complete set of excitation ground motions are reported in Figure 8.

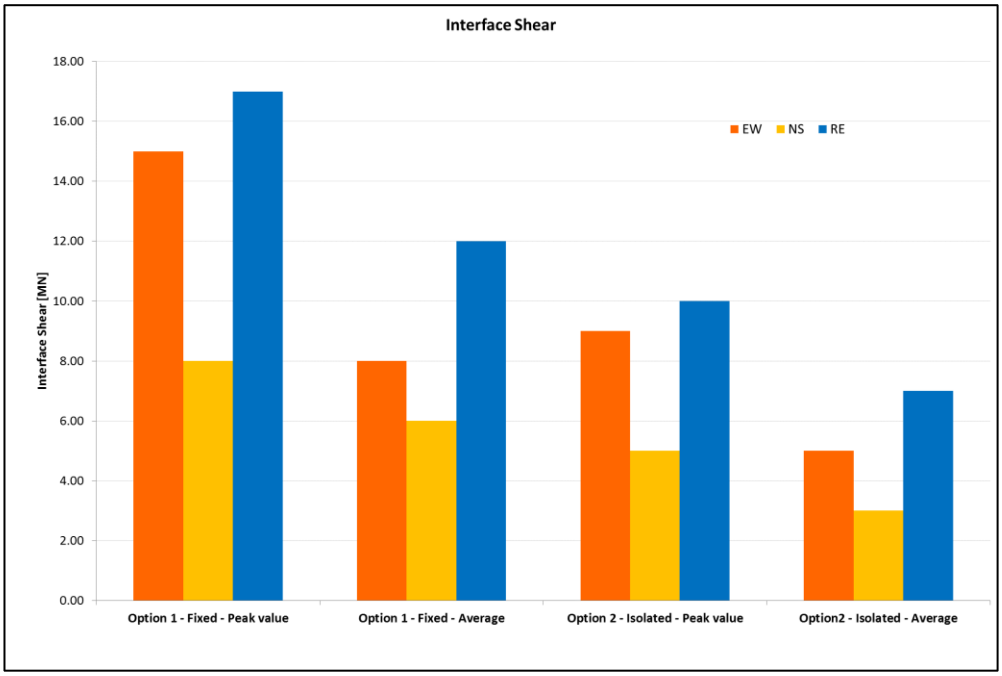

Figure 8: Shear forces at the conservatory/tower interface (EW – East/West, NS – North/South, RE – Resultant) – Peak and average values

When the conservatory is isolated at all towers, peak resultant shear forces at the interface are reduced by 40% at least, allowing significant material savings and design simplification of the supporting structure at the top of the towers.

Predicted factored FPB displacements/damper strokes

The design displacement for the FPB and dampers was calculated as the maximum resultant displacement at each tower, averaged over the complete set of ground motion time-histories and scaled by a safety factor of 1.5, as specified by the National Expert Panel for the project.

FPB/Damper location	Design displacement [mm]
T2	225
T3S	240
T4S	245
T5	200

Table 4: Maximum average resultant displacement – 1.5 Safety factor applied

Figure 9 shows the peak and average displacements scaled by the safety factor 1.5. Maximum displacement reaches 335mm at the T4 tower.

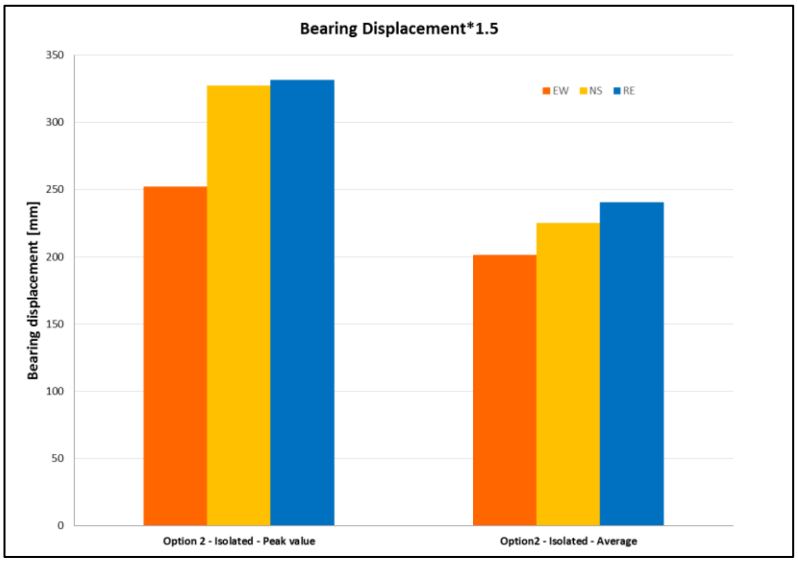

Figure 9: Bearing Displacement (EW – East/West, NS – North/South, RE – Resultant) – Peak and average values

These results suggest that the dimensions of the FPB and dampers could be reduced further (from values given in Tables 1 and 2). But some margins are currently maintained due to uncertainties about the conservatory final mass.

The predicted steel tonnage saving on the conservatory alone is estimated to 1322 t. Table 5 below provide the details of the assessment.

Structural member	Bidding phase mass [t]	Optimization [t]	Saving[t]	Reduction
Primary truss	2822	2200	622	22%
Secondary trusses	1425	1325	100	7%
Links	1042	692	350	33%
Transfer bridge	453	453	0	——
Bearing components	623	623	0	——
Lift slab and floor beam	500	250	250	50%
Total	6865	5543	**1322**	19%

Table 5: Steel tonnage reduction – Conservatory structure

Hybrid outrigger – Design principle

Outriggers in tall buildings are typically stiff links connecting the core structure to outer columns. They are designed to improve the building overturning stiffness and strength. Outriggers can effectively reduce building drift and core wind overturning moments. But generally, typical outriggers are steel structures and cost can be a limiting factor.

For the RCCQ North Towers, Arup design team proposed an innovative design combining a cost-effective concrete outrigger wall and a steel frame. During seismic events, part of the eccentrically-braced steel frame (EBF) is to act as a structural ‘fuse’ by yielding at controlled demand levels and limit the damage in the outrigger wall. This solution also simplifies the design of the wall-to-mega-column connection and offers significant cost savings.

Before prototype testing and physical validation of the design, extensive CAE simulations were performed to understand the structural behavior and fine tune the details of the hybrid outrigger (EBF fuse and reinforced concrete outrigger wall)

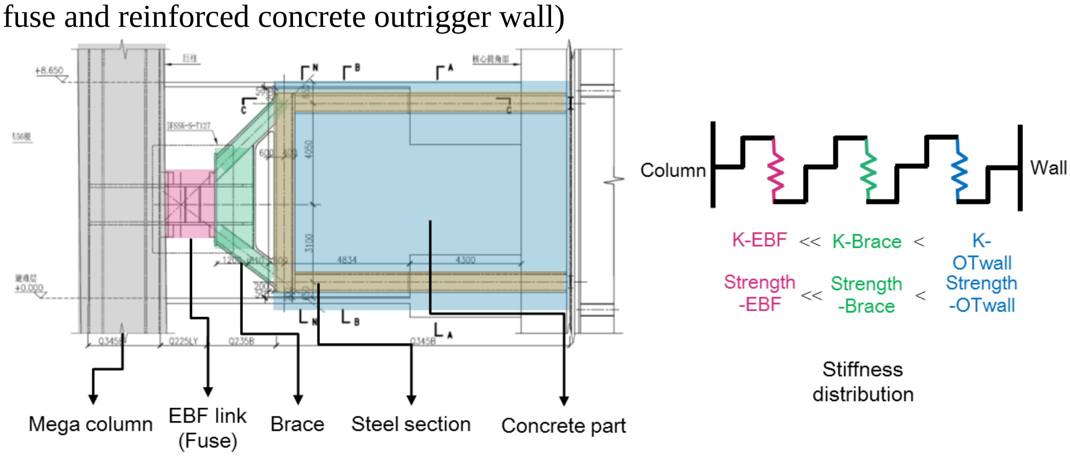

Figure 10: RCCQ Project – Hybrid outrigger concept © Arup

Detailed modelling of the fuse and outrigger structure

For the modelling of the steel fuse element and of the reinforced concrete outrigger structure, Arup used its well established, validated methods:

- Solid element mesh for the concrete components (mega-column, outrigger wall, sections of core walls)
- Shell element mesh for the steel frame and the fuse plates
- Beam elements for the embedded reinforcing bars
- *CONSTRAINED_LAGRANGE_IN_SOLID constraints between the rebar and the surrounding concrete mesh
- *CONTACT_TIED_SHELL_EDGE_TO_SURFACE constraints between the steel frame and surrounding concrete mesh

A detailed 3D model of the concrete sections was finely meshed (50mm elements in average) to accurately capture the geometry of the walls.

8-node and 6-node solid elements in LS-DYNA (*ELEMENT_SOLID with ELFORM=1 – Constant stress element) have been used for modelling the concrete.

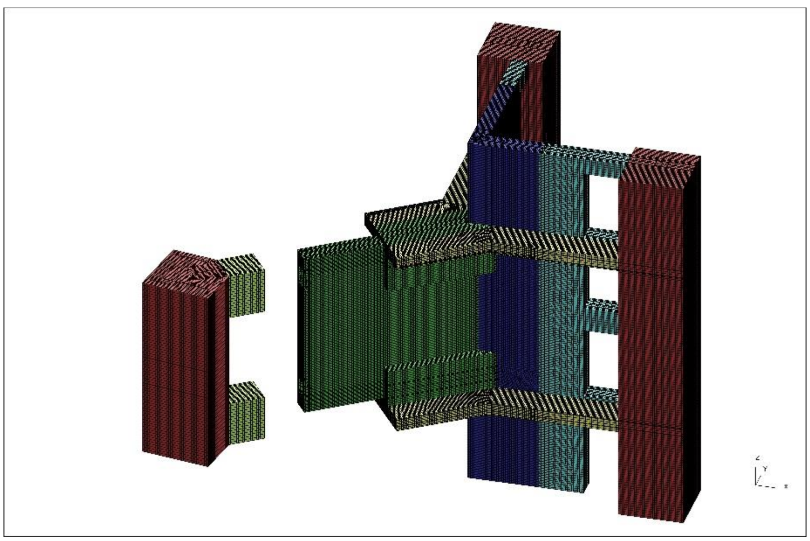

Figure 11: Concrete components – Detailed mesh

The reinforcing steel plates in the outrigger wall and EBT elements were modelled explicitly using shell elements (Element formulation Type 16- with NIP =10). Figure 12 below illustrates typical steelwork geometry and plate thicknesses.

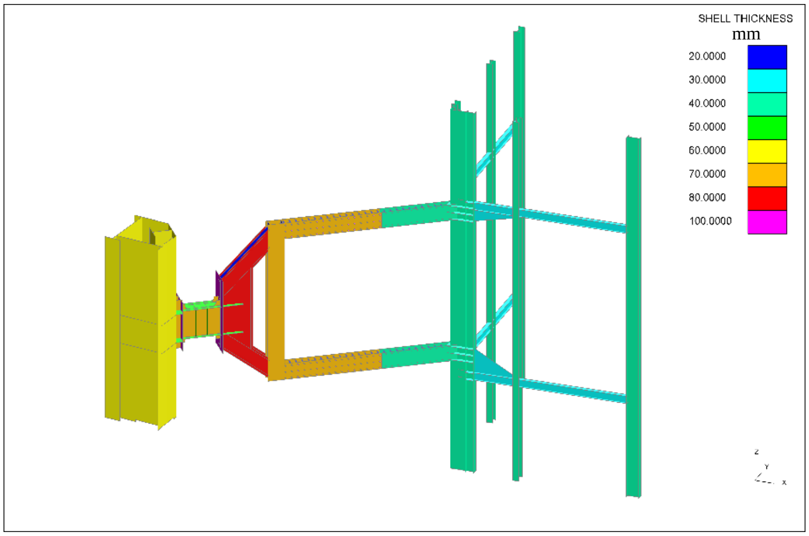

Figure 12: Steel components – Detailed mesh and thicknesses

The reinforcement bars were modelled explicitly with beam elements (*BEAM_ELEMENT formulation Type 1 – Hughes-Liu). Rebar components were defined as per bar diameters, varying from Ø14 to Ø36mm. The average element size of the rebar beams was adjusted to around 100mm to match the element density of the surrounding concrete.

The rebar beams were embedded into the adjacent solid concrete by means of a *CONSTRAINED_LAGRANGE_IN_SOLID constraint. Therefore shared nodes or compatible mesh between concrete and steel beams was not required.

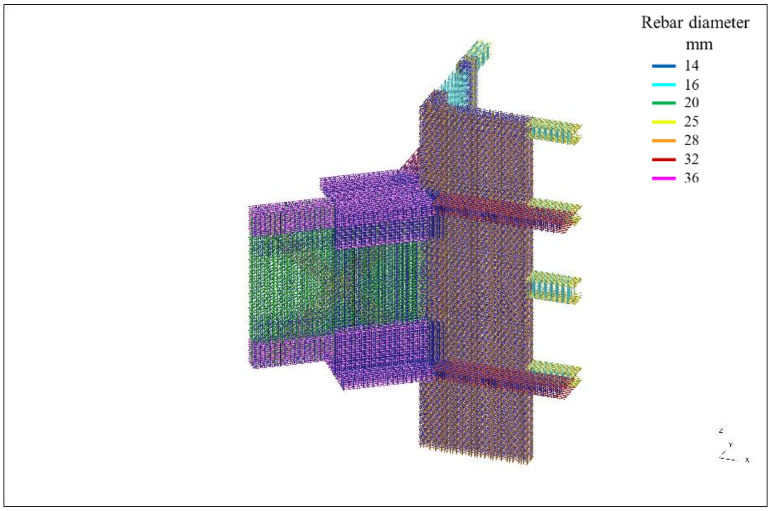

Figure 13: Rebar components – Detailed mesh and element diameters

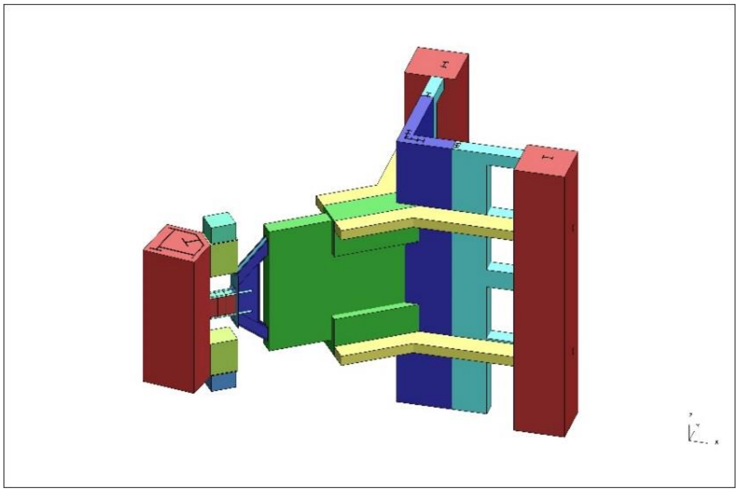

Figure 14: Hybrid OT – Complete analysis model

Material properties

- Concrete

The buttress concrete was specified as grade C60. The *MAT_WINFRITH_CONCRETE material in LS-DYNA is well suited to predict damage (crushing/cracking) and was used for all concrete parts in the models.

The concrete cube and tensile strengths, according to GB50010-2010[5], are listed in Table 6.

Components	Concrete Grade	Young's modulus [GPa]	Cube Strength [MPa]	Tensile Strength [MPa]
Concrete	C60	35	38.5	2.4

Table 6: Concrete Properties

The *MAT_WINFRITH_CONCRETE material model itself does not account for the increase in strength and strain due to confinement, but by modeling the reinforcement explicitly in the concrete, an increase of strength and ductility can be observed. The confinement effect is of course dependent on the rebar number and distribution.

- Steel plates

The steel plates were modelled with thick-shell elements and *MAT_PLASTIC_KINEMATIC. Their grade was Q235 and Q345, and the yield stress was dictated by GB 50017-2011 [6] as described in Table 7.

Components		**Minimum yield strength [MPa], Nominal thickness [mm]**				
Thickness	t	≤16	>16 ~ 40	>40 ~63	>63~80	>80 ~100
Q235	σ_y	235	225	215	205	205
Q345	σ_y	345	335	325	315	305

Table 7: Steel plates – Yield stress

- Reinforcement bar and shear stud steel

An elasto-plastic material was assigned to all rebar and shear stud beam elements. These elements used the *MAT_PLASTIC_KINEMATIC material model with a bilinear stress-strain characteristic curve.

The rebar and stud material properties are listed below in Table 8.

Components	Yield Strength f_{yk} [MPa]	Stiffness E [GPa]
Rebar	400/500	206
Shear studs	345	206

Table 8: Steel Rebar/Shear Stud Properties

Hybrid O/T simulation and validation

Push-over and cyclic loading analyses of the complete hybrid OT system were also performed. Simulations results compare well with the physical tests carried out by CABR.
In particular:

- For the load cases considered, the Hybrid OT system works as intended. The hysteretic response of the Hybrid OT system confirms its energy dissipation role.

- The fuse element fulfils its function. It yields early and concentrates the damage. The main outrigger steel frame remains elastic and the concrete wall is protected.

- When cyclic loading increases, cracks develop on the OT wall, but the main crack width is less than 0.5mm (minimal repair required). The rebars in the OT wall and ring beams remain elastic.

Figure 15 and 16 below show comparisons of force-deflection simulation v. test results for the push-over and cyclic loadings respectively.

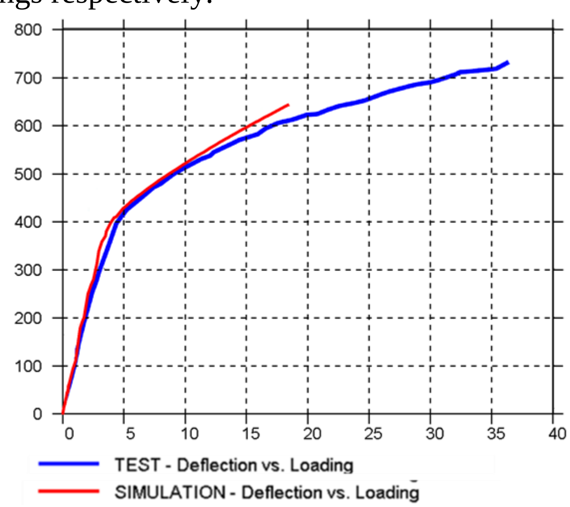

Figure 15: Hybrid OT system – Force-deflection under Push-over – Comparison to test

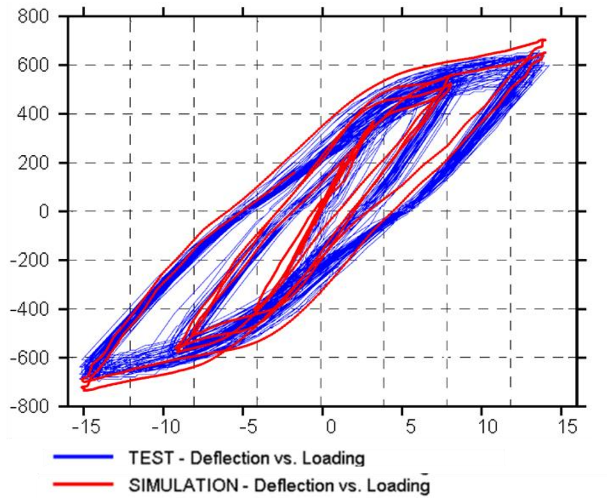

Figure 16: Hybrid OT system – Force-deflection under cyclic loading – Comparison to test

Both quasi-static push-over and cyclic simulations compare well with physical measurements. These detailed FE simulations have supported the design optimization phase as well as the physical prototype testing performed by CABR.

Figure 17 compares the FE prediction of concrete cracking in the Hybrid OT wall with physical testing results

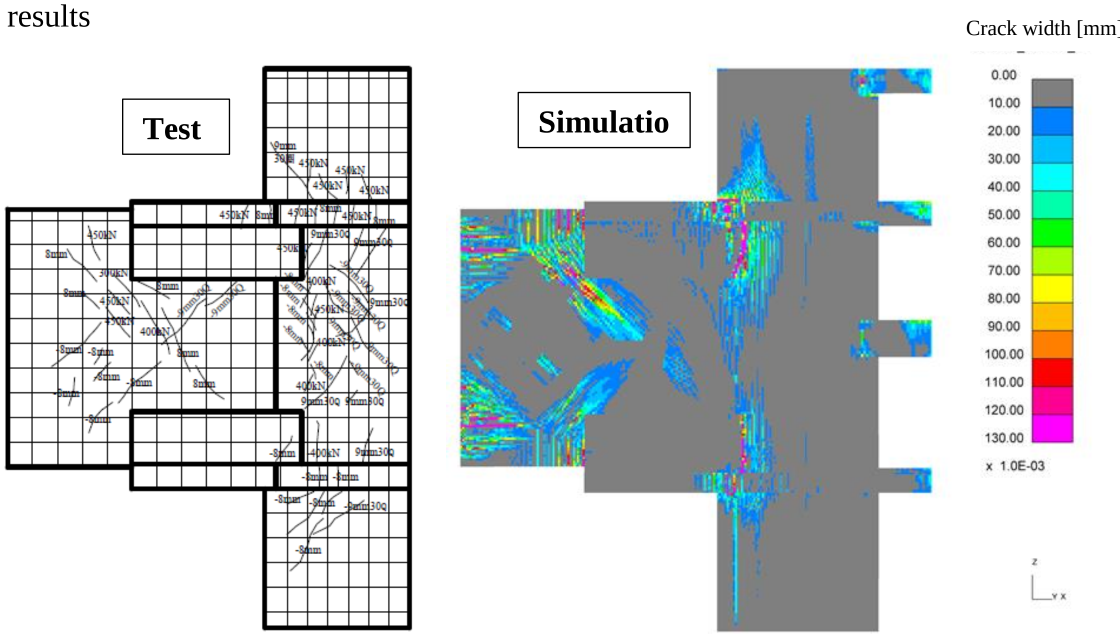

Figure 17: Hybrid OT system – Concrete cracking – FE simulation and cyclic test

Cracking distribution and extent are reasonably well predicted by the LS-DYNA simulation model.

Conclusions

By deploying advanced LS-DYNA capabilities, structurally reliable and cost-efficient options for the design of the Raffles City Chongqing were identified and validated. The innovative isolation scheme could meet the safety and sustainability targets of the programme and achieve the architecturally ambitious design within stringent budget constraints. Arup's technical proposal was reviewed and accepted by the Chinese Expert Panel and praised by all shareholders.

The LS-DYNA software was also chosen to perform the analysis mainly for its capabilities to represent geometric and material nonlinearities, in particular complex concrete cracking and crushing behavior, the possibilities to model contacts and concrete/rebar interactions.
A detailed model of the Hybrid OT wall was developed and used to optimize the design of the EBF link, steel braces and concrete wall detailing. Test protocol and set-up were validated and optimized before actual physical testing began. Further support was provided during the test campaign. This analysis work contributed to reduce the risks inherent to the prototype testing and was instrumental to guarantee its success.
The detailed FE model was validated against physical test results and best modelling practices were captured. This knowledge will be applied to future projects.
All tests performed so far have confirmed that the design is performing as intended. Especially under seismic loading, damage to the concrete wall remain minimal.

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