Osaka International Convention Centre

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1.
Aerial view of OICC against Osaka City centre.

Introduction

The Osaka International Convention Centre (OICC) in Nakanoshima, Osaka, Japan, opened to the public in March 2000. The building comprises five major facilities - a plaza, an event exhibition hall, an auditorium, a circular conference hall, and a variety of medium and small conference halls.

Due to the area restrictions of the site, these various components were planned vertically rather than horizontally. For a complex of this type, such a design solution is extremely unusual, and perhaps unique. As a result, the final design comprised six supporting structural cores, one at each corner and one at the midpoints of the long sides, with a total height of 104m.

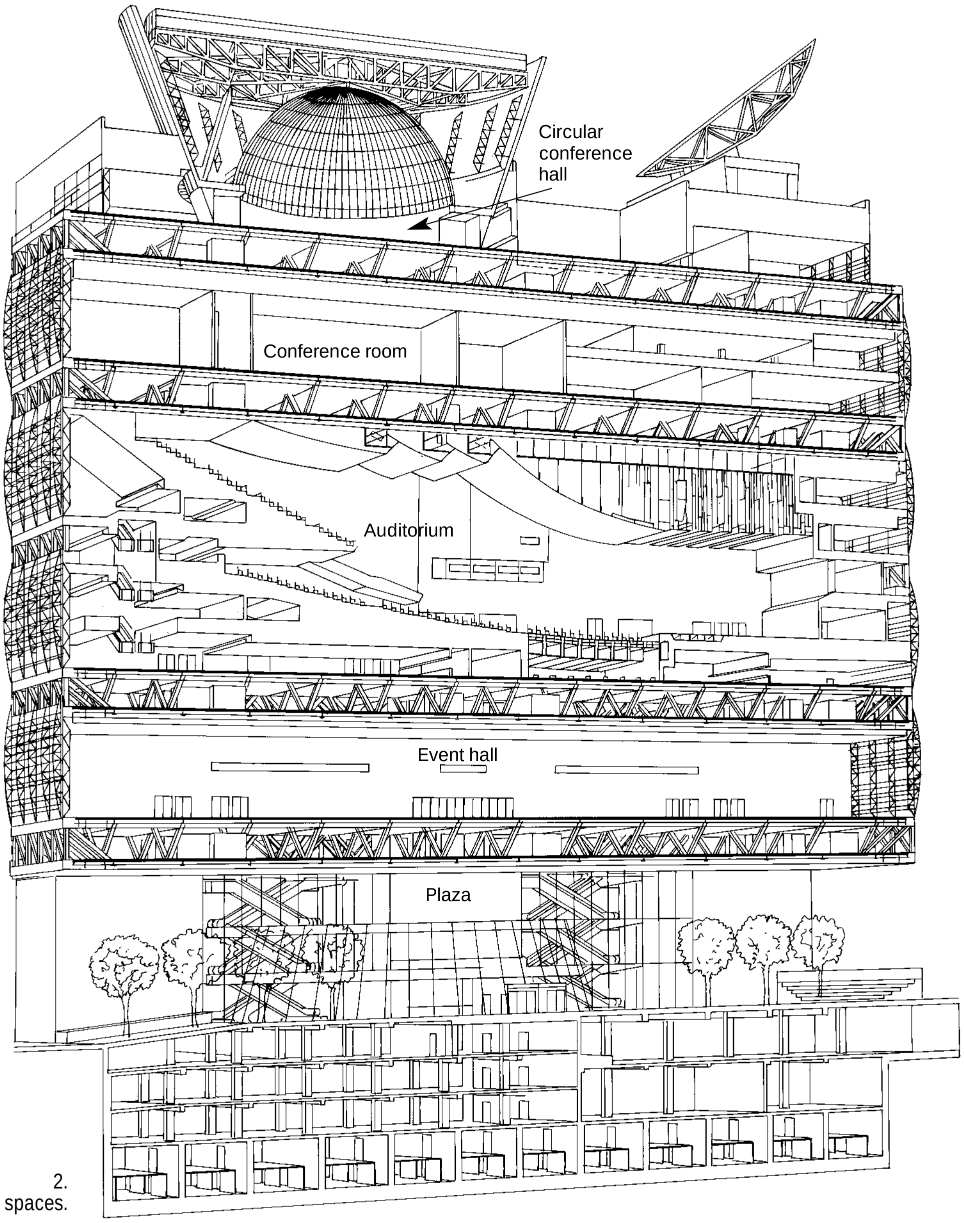
There are 13 levels, and spaces within the single-storey deep supertrusses at every third level are used for the mechanical, electrical, and public health installations. The building is rectangular in plan, 95m x 59m, and has a plot area of 6756m^2 and a total floor area of 67 545m^2. The budget was around US$500M.

The project originated from an architectural design competition held in 1994 and won by the Kurokawa Epstein Arup Consortium, comprising Kisho Kurokawa Architects & Associates, A Epstein & Sons International, and Arup in Japan as structural and seismic engineering designer.

For the project, Arup drew on its 'seamless' design approach. Various parts of the firm worldwide supported the Japanese office, including members from Hong Kong, London, and Birmingham, embracing structural, seismic, and geotechnical expertise. The structural and seismic design was reviewed by the Building Centre of Japan (BCJ) and approved by the Ministry of Construction - as is standard practice for Japanese buildings over 60m tall.

The OICC was the first Arup Japanese project to receive BCJ's technical appraisal for high buildings.

Arup's Japanese office supervised the site from the commencement of construction in November 1997. The project was built by a joint venture of 10 companies, led by Takenaka Corporation, Osaka.



2.
Schematic section through OICC showing principal internal spaces.

The building

The ground floor extends upwards for two of the 13 levels, and is a virtually column-free space; it includes a public open plaza with a 15.5m high ceiling and a circular stage 9m in diameter.

The event exhibition hall, from the third to the fifth levels, has a floor area of 2600m², which can be partitioned into two or three sections. Here, the floor loading intensity is 10kPa (1 tonne/m²), and the ceiling height is 9.4m.

Above this, the auditorium, accommodated within the sixth to ninth levels, is a theatre-type, multi-purpose hall, seating 2754. Its movable stage can be arranged in an end or centre configuration, and the entire auditorium can be partitioned into two.

On the 10th floor are conference rooms including one seating 600, though combinations can be made that accommodate up to 1000 people, thus creating a space suitable for use by international conferences.

Above again is the circular conference hall on the 12th floor; this is about 23m in diameter, with an area of 393m². Its domed ceiling rises from 4.6m to 16.8m. This spacious hall accommodates up to 550 people and features some of the most advanced, state-of-the art conference amenities and equipment. Finally, there is a heliport on the roof above this hall.

The structure

The six 14m x 12m structural cores at the corners and the midpoints on the long sides have concrete walls up to the first floor, 1.5m above ground.

The main frame of the superstructure consists of 1.2m x 1.2m steel H-section columns, with flanges and webs up to a maximum thickness of 80mm.

The high strength, heat-treated (quenched and tempered) steel has a tensile strength of 590N/mm². The steel of the beams and the one-storey deep 'supertrusses' has 520N/ mm² tensile strength.

At every third level, these supertrusses span between the cores, with intermediate floors either hung or propped from them to create column-free spaces. The structure utilises 'unbonded braces', a system of passive seismic energy absorbing devices developed by Nippon Steel Corporation. These provide hysteretic damping and limit the force levels generated in non-sacrificial structural elements.

Each of the six structural cores consists of columns, beams and unbonded braces. The cores are connected to each other by more sets of 20m long unbonded braces, spanning two floors and providing horizontal resistance.

The total weight of steelwork, including secondary steel, is approximately 34 000 tonnes.

3.
The completed building from the north east.

5.
The plaza at night.

6 below:
The auditorium.

4.
The event exhibition hall.

7.
The auditorium stage.

Kobe earthquake 1995

The Asia-Pacific region has high seismic activity; Japan is on the eastern edge of the European tectonic plate and bounded to the east and the south by the Pacific and Philippine plates.

The Kobe earthquake of 17 January 1995 had its epicentre on the north side of Awaji Island, only about 20km from the city, with a magnitude measuring M=7.2. The peak ground accelerations were large both horizontally and vertically, with a duration shaking of 10-15 seconds. The peak ground acceleration measured at Kobe Meteorological Observatory was 818gal (cm/sec²) or 0.8g.

Damage to steel structures from brittle fracture was reported, with the main source of the damage observed to be large inelastic deformations concentrated in column and beam ends, as well as cracks in or near welding sites.

At that time designers generally assumed that, in an earthquake, plastic hinges form in beams and thereby dissipate energy. This assumption became dubious, however, after results from the Kobe earthquake were examined. In many instances, connections did not behave in a ductile manner, and fractured unexpectedly.

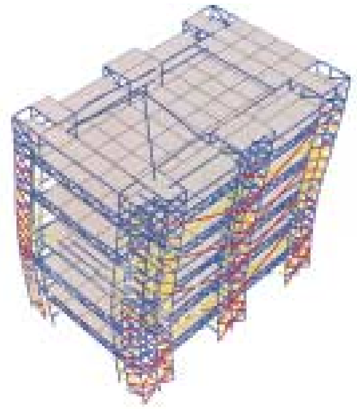
The ductility of materials is expressed as a yield ratio, a ratio of displacement at ultimate tensile strength to yield strength. However, no consideration had been given to material fracture toughness, and in the context of an earthquake, materials must have both a good yield ratio and fracture toughness to ensure ductile behaviour.

Arup's challenge in the seismic design of the main structural frame of the OICC was to achieve both damage control and ductile behaviour, whilst avoiding brittle fracture of connections.

Seismic performance-based design

In Japan, the basic seismic performance criteria for designing buildings which exceed 60m in height are outlined in a guidance paper issued in 1986 by the High Rise Building Appraisal section of BCJ[1].

Two seismic events, commonly referred to as 'Level 1' and 'Level 2', must be considered. The specific intensity of these events varies with geographical location, but qualitatively, 'Level 1' represents an event which may occur more than once in the lifetime of the building, while 'Level 2' represents the maximum intensity of seismic event which has occurred at the site in the past or which may possibly occur in the future. In turn, the performance of the structure under a Level 1 event is limited such that *'... the building shall not be damaged and the main structure shall behave within its elastic limit...*' while for a Level 2 event '*... the building shall not collapse, or cladding fall, etc, such that there is a threat to human life.*'



8.
LS-DYNA 3D model
showing OICC's seismic performance.

However, in the aftermath to the damage observed at Kobe, the performance criteria were redefined, together with the inclusion of two additional design events, 'Level 3 earthquake' and 'active fault effect', as follows:

'The building should be fully operational under a Level 1 event, represented by an earthquake with a peak ground velocity of 20kine (cm/sec):

- *no damage to structural elements*
- *plasticity only to be permitted in the unbonded braces*
- *no damage to non-structural elements*
- *storey drifts limited to less than 1/200* ['storey drift' is the relative horizontal displacement between the upper floor and the floor of each storey.]

The building should remain operational under a Level 2 event, represented by an earthquake with a peak ground velocity of 40kine (cm/sec):

- *damage to be light, requiring minor repair*
- *beams permitted to form plastic hinges*
- *no plastic hinges permitted in columns*
- *storey drifts limited to less 1/100*
- *storey displacement ductility limited to less than* $\mu_\Delta = 2.0$.

(μ_Δ: *frame yield displacement of storey)*

Following a 'Level 3' earthquake, defined by a peak ground velocity of 60kine (cm/sec), the building should ensure the life safety of its occupants:

- *damage to be moderate, requiring repair*
- *some building systems to be protected.*

Under the 'near active fault' phenomenon, characterised by a single impulse with a peak ground velocity of 80kine (cm/sec), collapse prevention should be achieved:

- *structural collapse should be prevented*
- *non-structural elements may fail.'*

3D non-linear finite element time history analysis

To simulate the performance of the building during a large earthquake, Arup carried out several three-dimensional finite element time history analyses using LS-DYNA 3D. This advanced software is more commonly used to model highly complex non-linear behaviour, such as collisions in the automotive industry and virtual prototyping of fuel flasks in the nuclear industry.

The OICC project, however, represented the first major civil engineering application of LS-DYNA 3D. Another program, NASTRAN, was utilised for all linear design check analyses, while an LS-DYNA model, incorporating 10 000 non-linear elements to capture the potential inelastic behaviour of all structural members, was developed in parallel to validate the non-linear seismic performance of the building. Ground motions, comprising horizontal and vertical components with standardised peak ground velocities of 20, 40, and 60kine, together with a pulse signal representing the potential near fault phenomena of the active Uemachi Fault in Osaka city, constituted the suite of input time histories for validation of the seismic performance.

Part of this suite included the Fukushima (N-S) signal, recorded in the free field close to the site during the 1995 Kobe earthquake. However, to assess the significant soil / structure interaction in the deep piled basement, input signals for the time history analyses were applied at the base of the piles. To ensure consistency with the Fukushima free field site response, it was necessary to deconvolve this signal to the base of piles level. These site responses were assessed by Arup geotechnical specialists in Hong Kong using the program SIREN, which analyses the response of a one-dimensional soil column when subjected to an earthquake motion input.

'Damage-tolerant' design

'Unbonded braces' are passive devices which absorb seismic energy efficiently during an earthquake. In the 'damage-tolerant' approach to design adopted for the OICC, these braces are sacrificial elements designed to leave the rest of the building with little damage from an earthquake. Inside the unbonded braces are flat or cross-shaped steel braces, covered with debonding chemicals, that can stretch and shrink freely under seismic loads but will not buckle, since lateral support is provided by concrete filled tubes surrounding the braces. The steel grade used for the braces has a minimum yield point of 235N/mm², and tensile strength of 400-510N/mm². To control the seismic energy-absorbing performance of steel braces, an upper boundary of 295N/mm² to the yield point was additionally specified. The maximum steel brace dimensions are 40mm x 700mm, inside a concrete-filled 800mm x 650mm steel casing. The unbonded braces can absorb 40% to 75% of the total seismic energy through the time history analysis, effectively reducing the energy input to the building.

9.
An unbonded brace before erection.

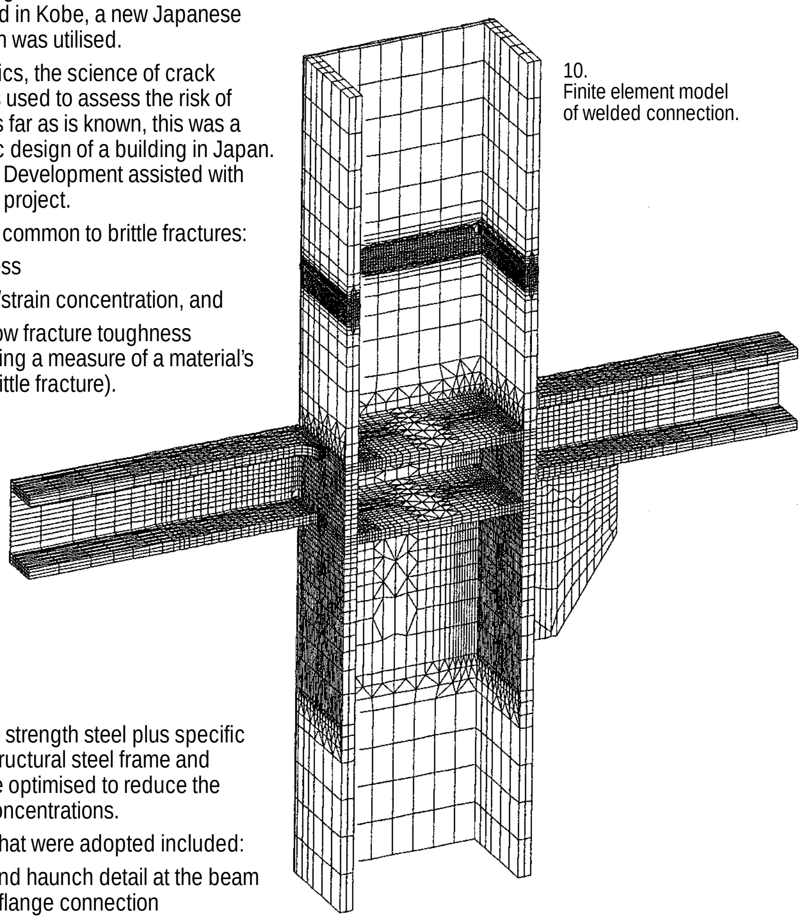
Ductility and fracture toughness

To control damage to the main superstructure and ensure that the steel behaved in a ductile manner without developing the brittle fractures that had been encountered in Kobe, a new Japanese steel specification was utilised.

Fracture mechanics, the science of crack propagation, was used to assess the risk of brittle fracture. As far as is known, this was a first in the seismic design of a building in Japan. Arup Research & Development assisted with this aspect of the project.

Three factors are common to brittle fractures:

- high tensile stress
- points of stress/strain concentration, and
- materials with low fracture toughness ('toughness' being a measure of a material's resistance to brittle fracture).



10.
Finite element model of welded connection.

The use of higher strength steel plus specific detailing of the structural steel frame and connections were optimised to reduce the effect of stress concentrations.

Specific details that were adopted included:

- the use of a round haunch detail at the beam flange / column flange connection
- removal of run on / run off tabs (these had previously been left in place)
- prohibition of temporary attachments.

Brittle fractures had initiated from both of the latter details in the Kobe earthquake.

The required fracture toughness was established using the principles described in *PD6493*[2] and *WES2805*[3], which both describe methods for assessing the acceptability of flaws in welded structures. The input requirements for a fracture assessment include:

- flaw geometry, size, and location
- stresses, primary and secondary
- material properties.

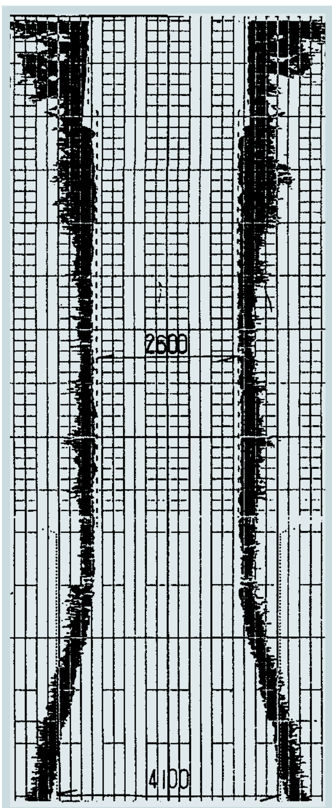
To assess toughness requirements, an assumed flaw geometry was adopted[4]. The stress condition for a typical supertruss column connection was established using a 3D non-linear finite element time history analysis. This was validated using a full-scale mock-up, which was also used to establish residual stress levels and prove the welding procedure.

Toughness requirements were specified in terms of both Crack Tip Opening Displacement (CTOD) and Charpy impact energy. Material properties were specified for both parent and weld metals.

Soils and foundations

The topsoil, alluvial sand, and clay lie within 29m depth from ground level (GL-29m. The diluvial sand, with SPT (standard penetration test) readings of 34-60, and clay with SPT of 13-20, lie alternately below GL-29m to GL-76m. The typical groundwater levels were reported to be around GL-8m.

The foundations consist of cast in situ concrete bored piles with under-reams bearing on the diluvial gravel at GL-53m. In the seismic design of the piling for OICC, non-linear pile / ground interaction studies were carried out by Arup geotechnical specialists in the Hong Kong office.



11.
Scan of Koden borehole test.

The piles typically have a 2.6m diameter shaft with a 4.0m diameter under-ream. A proprietary *Koden* ultrasonic wave test scanned and checked the bored hole before the concrete was poured. For the first time in Japan, the quality of the cast in situ concrete piles was inspected by the pile integrity test, which measures the velocity of an ultrasonic wave transmitted from two probes inside pile concrete.

The excavation extended to a depth of 18.5m, and this resulted in the strata beneath heaving, due to the release of pressure from above. It was assumed that granular strata would heave immediately but that the heave in the clay strata would continue for some time; thus, after the basement was built, the slab would be subject both to uplift forces from water pressure and heave-generated pressure from the soil. An alternative to allowing this soil pressure to reach the slab was to construct the slab on a collapsible material with the following properties:

- It must be able to support the weight of the concrete slab before it hardens.
- It must collapse at a known pressure.
- It must continue to collapse until the remaining heave is complete.
- It must not degrade in a manner that produces dangerous gases such as methane.

To calculate the extent of the 'rebound', the heave was analysed using the program VDISP, with collaboration from Arup geotechnical specialists in the UK. If the space beneath the slab was filled with appropriate collapsible material, the analysis indicated that it should be able to accommodate the heave. Collapsible boards with the required properties were therefore placed under the basement slabs.

The retaining wall to the basement was the composite basement 'TSP' wall developed by Takenaka Corporation. During the excavation, soil-cement pile walls were used as earth retaining walls, with H-section steel beams - which are usually buried after construction completion - forming temporary reinforcing beams for them. The basement exterior walls consist of vertically installed H-section beams with studs that embedded within the concrete walls. The composite action thus developed between the H-section and reinforced concrete wall effectively reduced the overall thickness, resulting in a more efficient use of the site.

Top-down construction sequence

This procedure, which allowed excavation of the basement and erection of the steelwork for the superstructure simultaneously, was implemented in view of the fact that not only was the schedule tight, but it involved both a time-consuming deep excavation and limited space for site storage.

The 'soil mixing' (soil, cement and bentonite) retaining wall was constructed first, and then the piles placed from ground level downward. A borehole was constructed using the bentonite and a drilling bucket with steel casing on the borehole top. The under-ream was installed using an 'earthdrill' machine, and the concrete placed using tremie pipe. A basement steel section encased in SRC column, known as 'Koshinchu', which transfers the construction loading to the pile foundation during steel erection, was pushed into a pile top from ground level as soon as the concrete casting was completed. Excavation of the basement and erection of the steel superstructure were carried out after completion of the ground floor slabs.

Soil rebound was monitored and continuously compared with predictions from analysis throughout the duration of construction.

This monitoring was done using a measurement system set in the bored holes, installed before commencement of construction.

Site steel erection

This was done in the following sequence, using a 35 tonne capacity tower crane in each of the four corner cores and a 20 tonne crane in each of the two cores midway on the long sides:

(1) The steel cores were erected.

(2) The longitudinal supertrusses were assembled at ground level or at a lower supertruss floor level, and lifted up by tower crane.

(3) The unbonded braces spanning two floors were lifted and connected to their respective pairs of cores.

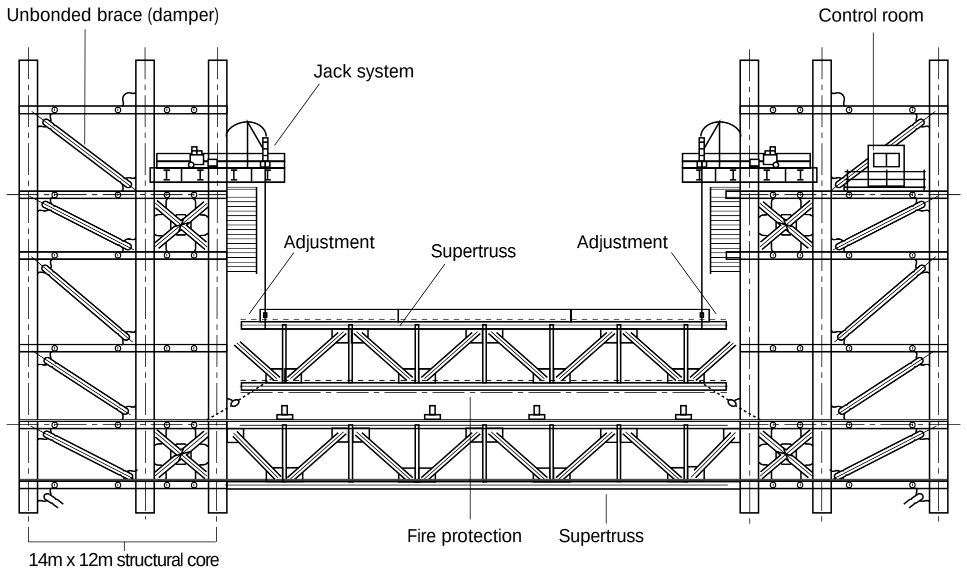
(4) A 34m x 95m supertruss floor, consisting of 15 transverse supertrusses plus secondary beams, divided into two or three parts, was assembled at ground level or a lower supertruss floor level.

(5) The metal decks with fire protection were constructed, and the mechanical, electrical, and public health facilities installed.

(6) Each resulting floor block, with a maximum weight of approximately 840 tonnes, was jacked up from the upper supertruss level. The lifting speed averaged 2m - 3m/hour.

(7) The joints for the columns within the cores were temporarily connected during jacking. To control the column axial forces to meet the design criteria, these were released once and then connected again after the jacking.

During construction pre-erection analyses were carried out to check the structural stability. The axial forces acting on the columns were also gauged.



12 below left:
OICC under construction from north west, August 1998.

13 above:
Construction sequence for lifting a supertruss.

14 below centre:
CO_2 gas shielded metal arc welding of a super column.

15 right:
Lifting a supertruss.

16 bottom right:
Construction progress by November 1998.

17.
The auditorium under construction.

18.
Oil cylinder for fuse system.

19.
Tuned mass dampers.

20.
The auditorium in use.

Fuse system and tuned mass dampers for the auditorium

The auditorium structure had to fulfil two conflicting requirements.

- It had to be flexible enough to accommodate 'storey drift' of the main superframe whilst transferring seismic shear forces from the auditorium to the main frame during an earthquake.
- It had either to be stiff enough to limit vibrations under people load, or be highly damped.

To minimise the stress concentration from the sloping concrete slabs of the auditorium through the diaphragm action caused by the imposed storey drift of the main superframe in an earthquake, a unique system was adopted.

A slide system with seismic sensors and oil cylinders at the cantilevered tip of the second auditorium level (2F) was used as the electrical fuse system. When the sensors detect a seismic wave, the sliding system at 2F is automatically released to avoid the stress concentration - and the system is designed to recover after an earthquake. The 1F structure has longitudinal slits on the concrete slabs, which can provide in-plane stiffness reduction.

Also, tuned mass dampers (TMDs) are used to reduce vertical response resulting from audience movement during rock concerts. Special studies were made to improve performance of the structure against vibration from audience activities, various dynamic inputs being modelled and compared with criteria in published literature researched by Arup's Advanced Technology Group (ATG).

The dynamic response of the main auditorium structure under audience load from various activities - dancing, bouncing, jumping - was analysed by ATG; the maximum allowable acceleration from these was based both on a literature study and experience, and set to 10% of gravity acceleration (0.1g). For the upper structure, eight 3.5 tonne TMD units were needed at the cantilever tip next to the grid line and four 3 tonne TMD units at the back span. For the lower structure, 16TMD units of 2.5 tonnes to 3 tonnes were needed. Using TMDs reduced the response of the upper and lower structures from a maximum 0.4g to about the target 0.1g. Performance tests under cyclic load by impact machines and 50 persons confirmed that by using TMDs the vertical response was reduced by 30- 60% as opposed to that of not using TMDs.

Conclusion

OICC was a great challenge to Arup Japan, and proved a successful collaboration between various parts of Arup worldwide and the other design team members. The building was the first major project in Japan for Arup since Kansai International Airport[5], and demonstrated the firm's capability in advanced seismic design.

The Centre has been used not only for international and domestic conventions but also for musical concerts, art and flower arrangement exhibitions, academic symposiums, commercial exhibitions, etc, since March 2000, and has been dubbed 'Grand Cube' as a symbol of Osaka. Sadly, the city did not host the G8 Summit Meeting 2000 (OICC was designed to be its venue), but Osaka is now in the running for the 2008 Olympics.

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