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Applications of Hysteretic Steel Dampers in Buildings of Novel Feature

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ABSTRACT :

Situated in a highly active seismic region, Japan is abundant with innovative seismic control systems from base isolation to various passively-controlled systems utilizing devices such as viscous dampers and steel hysteretic dampers. Although the applications of these devices are often located within core framing and hidden from sight, at times the dampers are revealed and incorporated into the design structurally and architecturally. This paper will discuss a project which exhibits a novel application of steel hysteretic dampers, Osaka International Conference Centre (OICC). Albeit a project that was engineered nearly a decade past, the engineering techniques applied to realize this structure perhaps can be considered advanced even by today's standards.

KEYWORDS:	Hysteretic Damper, Unbonded Brace, 3D Non-linear Time History Analysis, LS-DYNA, Japan




1. INTRODUCTION

In 1994 an architectural design competition was held for a new convention center in Osaka, Japan. Kurokawa Epstein Arup Consortium's (Kisho Kurokawa Architect & Associates, A Epstein & Sons International, and Arup Japan) unique approach of solving the area restrictions of the site by planning various components vertically instead of horizontally proved to be victorious. Opened to the public in March 2000, the Osaka International Convention Centre (OICC) in Nakanoshima, Osaka, consists of five major components –a plaza, an event exhibition hall, an auditorium, a circular conference hall, and a variety of medium and small conference halls.

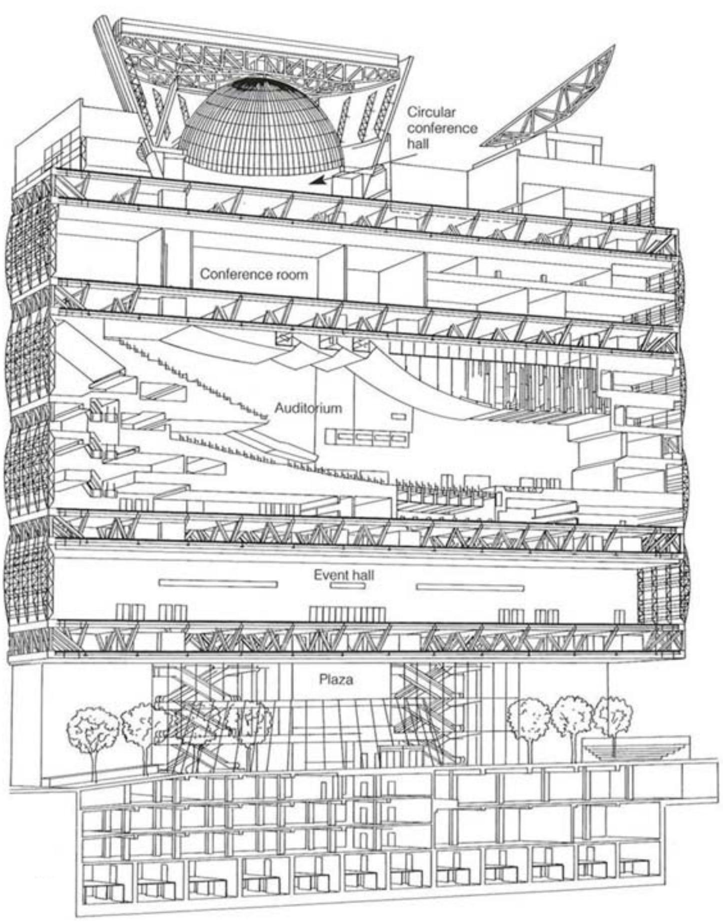

Figure 1: Schematic Section

A rectangular footprint of 95m x 59m, the building comprises of 13 levels extending to a height of 104m with a total floor area of 67,545m^2.

The ground floor is a virtually column-free space extending upwards for two levels, and includes a public open plaza with a 15.5m high ceiling and a circular stage. The third to fifth levels house the event exhibition hall of 2,600m^2 floor area, which can be partitioned into two or three sections. Here, the floor loading intensity is 10kPa and the ceiling height is 9.4m. The sixth to ninth levels consist of the auditorium and the multi-purpose hall, which seats 2,754 people which can be used in various forms by moving the stage, wall, and seats. Conference rooms located on level 10, can seat a maximum of 1,000 people in a single room by combining several separate rooms. Above all of this is the conference hall on the 12th floor, with a domed ceiling of 16.8m, and seating 550 people in 393m^2 floor area. Finally, on top of this dome, situates a heliport. See Figure 1.

2. SUPER-FRAME SYSTEM

To realize the unique spatial layout and requirements of the OICC, a super-frame scheme comprising of 'super-cores' and 'super-trusses' was implemented. Six 14mx12m super-cores are located at the four corners and the midpoints along the long direction of the building. Single-storey deep super-trusses span the great lengths to connect together the super-cores and create large column-free spaces throughout the building (Figure 2). The space within the storey high super-trusses is utilized as a plant room for various MEP equipments.

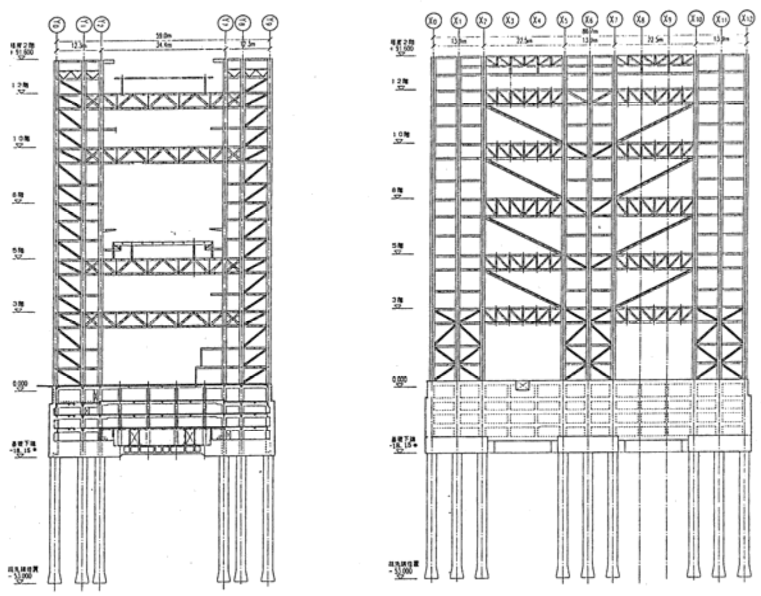

Figure 2: Sectional Diagrams of Structure

The super-cores are made up of 1.2m x 1.2m built-up steel H-shaped (I-shaped) sections with maximum plate thickness of 80mm. Typical member size of the super-truss elements are built-up steel sections with dimensions of 500mm x 500mm. The steel utilized is higher strength than conventional steel, with tensile


Figure 3: View from NE

strengths being greater than 590 N/mm^2 for the superstructure members.

A network of passive control devices called ‘Unbonded Braces’ developed by Nippon Steel Corp., were placed within the superstructure framing to provide supplementary damping and to limit the forces induced into the main structure. A detailed look into this device and analyses will be included in the following sections.

As seen in Figure 3, the key structural elements such as the super-frame and damping devices are clearly expressed in the exterior of the building – structural design was one of the key components of the architectural intent.

3. SEISMIC PERFORMANCE BASED DESIGN

The epicenter of 1995 Kobe earthquake magnitude 7.1 was only 60km from the site. The peak ground accelerations were large both horizontally and vertically – PGA of 818 cm/sec^2 was recorded at Kobe Meteorological Observatory. Numerous structural damages caused by steel brittle fractures around the column/beam ends and welding works were reported. In many instances, connections did not behave in the intended ductile manner and fractured unexpectedly. As such, the conventional design assumption of plastic hinges forming under larger earthquakes to dissipate energy without a detailed examination of the connection points was questioned. The design phase of OICC took place in a time when the field of structural engineering was still recovering from the effects of this devastating earthquake.

Building Code of Japan stipulate that any building exceeding 60m in height require a expert panel review process to be undertaken following suggested guidelines. Conventionally two seismic levels commonly referred to as Level 1 and Level 2 must be considered to validate the design. Level 1 represents a medium level earthquake, and Level 2 is a large earthquake with a return period of approximately 500yr. However, following the damage observed from the Kobe earthquake, additional performance criteria were stipulated, ‘Level 3 earthquake’ and a single impulse with a peak ground velocity of 80 cm/sec named ‘near fault effect’. Note, stipulation of such design criteria is uncommon even today. The criteria for OICC seismic design are summarized in Table 1.

Table 1: Special Design Criteria Stipulated for OICC *Note, most current guidelines stipulate different PGV values

Seismic Level	General Criteria
Level 1 PGV of 20 cm/sec	• building should be fully operational • 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
Level 2 PGV of 40 cm/sec	• building should remain operational • damage to be light, requiring minor repair • beams permitted to from plastic hinges • no plastic hinges permitted in columns • storey drifts limited to less 1/100 • storey displacement ductility limited to less than Δu=2.0
Level 3 PGV of 60 cm/sec	• building should ensure life safety of occupants • damage to be moderate, requiring repair • some building functions to be protected
Near Active Fault PGV of 80 cm/sec	• structural collapse to be prevented • non-structural elements may fail

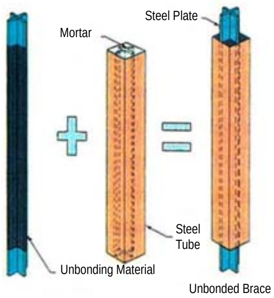

Figure 5: Unbonded Brace Diagram

As such, to meet onerous design targets, a 'damage-tolerant' design approach was adopted. Specifically, this approach entails placement of sacrificial elements within the structure that yield and absorb seismic energy to protect the rest of the building from damage. As for these sacrificial elements, a type of steel hysteretic damper, 'Unbonded Braces' developed by Nippon Steel Corp. was selected because of its fitting image of being part of a 'braced' super-frame structure.

These Unbonded braces consists of a core material that are flat or cross-shaped steel braces, covered with debonding chemicals that can stretch and shrink freely under seismic loads. Lateral buckling of the steel braces is constrained by the mortar encased steel tube (See Figures 5 and 6). The steel used for the core braces have a minimum yield point of $235N/mm^2$ and furthermore an upper boundary of $295N/mm^2$ was specified to ensure that variation of material property was minimized and the damping performance of the products was stabilized. The maximum Steel brace dimensions are 40mm x 700mm, inside a concrete-filled 800mm x 650mm steel casing.

As mentioned, Unbonded braces act as energy absorbing dampers by taking advantage of the elastic-plastic hysteretic curve which it experiences under large cyclic axial loading (see Figure 7).

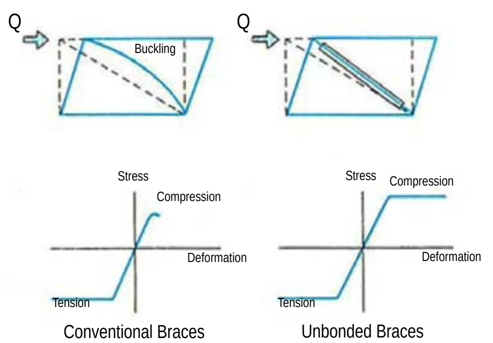

Figure 6: Conventional vs. Unbonded Braces

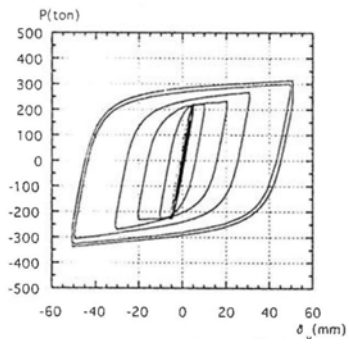

Figure 7: Unbonded Brace Hysteresis Curve

4. 3D NON-LINEAR FEM TIME HISTORY ANALYSIS

In recent times, it is common practice that the non-linear property of a building is determined in a floor by floor basis and converted into a stick model to carry out a time history analyses. A three dimensional time history analysis that models the non-linear property of dampers is rare – one that also models the elastic-plastic hysteretic behavior of the main structure is extremely rare. Due to the circumstances of seismic engineering in the wake of the Kobe earthquake and special nature of the structure, a three dimensional analysis model that simulates the potential elastic-plastic behavior of all structural elements was adopted for the time history analysis for the OICC.

The analysis software LS-DYNA3D was used to perform the three-dimensional finite element time history analyses. This advanced software is more commonly used to model highly complex non-1inear behavior used to perform crash worthiness analysis of automobiles and prototyping of nuclear fuel flasks. The OICC project was the first major civil engineering application of LS-DYNA3D. Consisting of approximately 10,000 non-linear elements, the

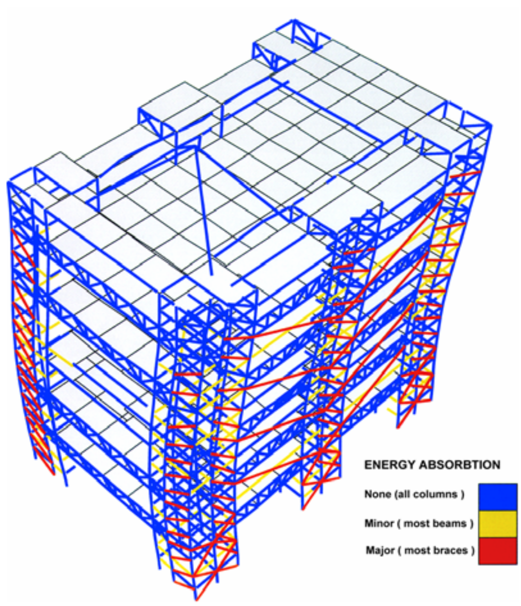

Figure 8: LS-DYNA Model

LS-DYNA model was created to capture the potential inelastic behavior of all structural members of the building (see Figure 8). In addition, NASTRAN was utilized in parallel to verify the linear design check analyses. Three-dimensional seismic ground motions with standardized peak ground velocities of 20, 40 and 60 cm/s, along with a pulse motion to simulate the 'near fault' phenomena of the active Uemachi Fault in Osaka city, were the input time history motions for the analytic verification of the building performance. To assess the soil/structure interaction in the deep piled foundation system, input signals for the time history analyses were applied at the base of the piles.

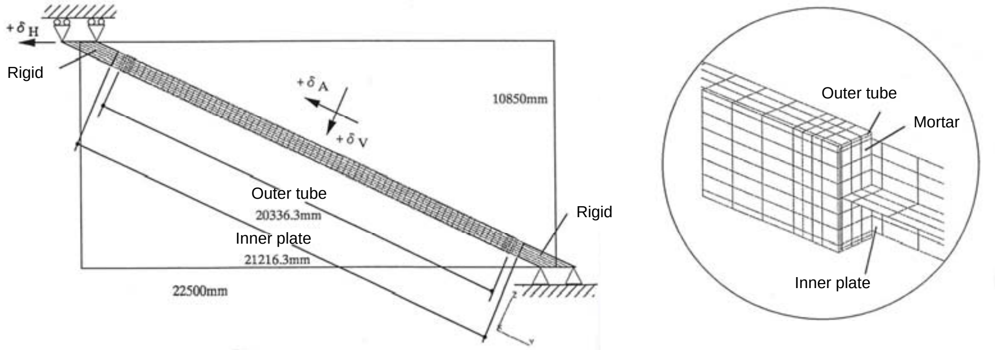

Figure 9: Unbonded Brace Analysis Model

Since Unbonded braces with lengths exceeding 20m are used, the effects of self load and distribution of axial strain were considered. The FEM modeling of the Unbonded braces is shown in Figure 9. Modeling assumptions of these elements include the following.

1. Inner steel plate will not deform out of line, and is symmetric about the center.
2. Both material nonlinearity (plasticity, cracks, compression failures) and geometric nonlinearity are considered.
3. Inner steel plates and joints are modeled as two-dimensional 'shell' elements, and steel pipes and mortar as three-dimensional 'solid' elements.
4. In order not to underestimate maximum bending forces at the end of the braces, the lower joints are rigidly connected and upper joints are rigidly connected except for one translational direction.
5. Joints are considered as rigid elements, and the Young's modulus for this part is modeled with 100 times that of the actual material.
6. Inner steel plates and the surrounding mortars are tied only in direction perpendicular to the axial direction, and can move freely to each other in the axial direction.

5. ANALYSIS RESULTS

Non-linear FEM time history analysis results are listed in Table 2, and Figures 10 to 12. For Level 1 earthquake, the main structure remains elastic whilst the Unbonded braces undergo plastic deformation with a maximum ductility factor of 2.75. Maximum storey drift ratio of 1/225 meets the h/200 target. At Level 2, the main structure also experiences plastic deformation, specifically plastic hinges form at the beam ends with a maximum ductility factor of 1.67 for the main structure. This satisfies general guidelines in Japan regarding ductility factors for the main structure which suggests less than 4.0 for a specific element and less than 2.0 as a storey behavior for a Level 2 earthquake. Again, the storey drift criteria of less than h/100 are met. Even under a large Level 3 earthquake, there are no plastic hinges formed in the columns. Both ductility factors and storey drift are well within acceptable ranges. Finally the 'near fault' strong pulse will cause hinges to form at the column base but as targeted, building collapse is prevented.

As observed in the energy dissipation plot (Figure 13), the Unbonded braces absorption of seismic energy is 75% in longitudinal direction and 45% in transverse direction – resulting in a significant reduction in the energy input to the super frames.

The Unbonded braces experience a combined stress of the bending moments induced by up to 20m of self-weight and the push-pull forces from earthquake loading. As shown in Figure 14, the elements were verified for general stress distribution and high localized stresses spots.

Table 2: Time History Analysis Results (Ductility Factors and Storey Drift)

SEISMIC SCALE	DIRECTION	MAIN FRAME DUCTILITY FACTOR	UNBONDED BRACE DUCTILITY FACTOR	MAXIMUM STOREY DRIFT	REMARK
LEVEl 1	X	LESS THAN 1.0	2.75	1/225	ELASTIC
	Y	LESS THAN 1.0	1.4	1/263	ELASTIC
LEVEL 2	X	1.25	3.75	1/158	PLASTIC HINGES AT BEAM END
	Y	1.67	3	1/121	PLASTIC HINGES AT BEAM END
LEVEL 3	X	1.75	4	1/122	PLASTIC HINGES AT BEAM END
	Y	2.7	5.3	1/74	PLASTIC HINGES AT BEAM END
NEAR ACTIVE FAULT	X	3.6	10.2	1/47	PLASTIC HINGES AT BEAM END
	Y	3.2	6.8	1/62	PLASTIC HINGES AT BEAM END AND COLUMN BASE

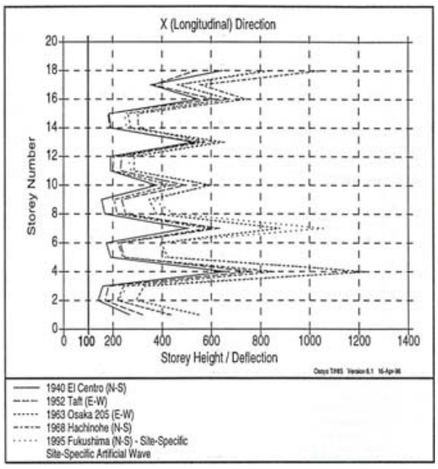

Figure 10: Horizontal Displacement

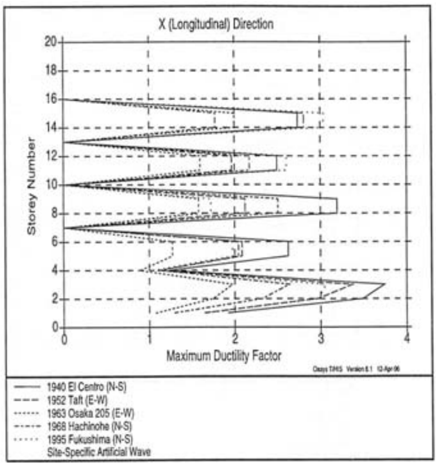

Figure 11: Ductility Factor of Unbonded Braces

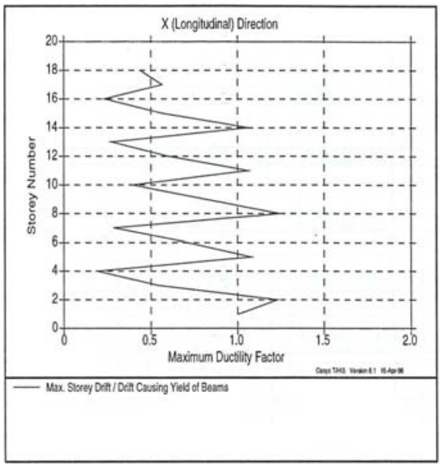

Figure 12: Ductility Factor of Main Structure

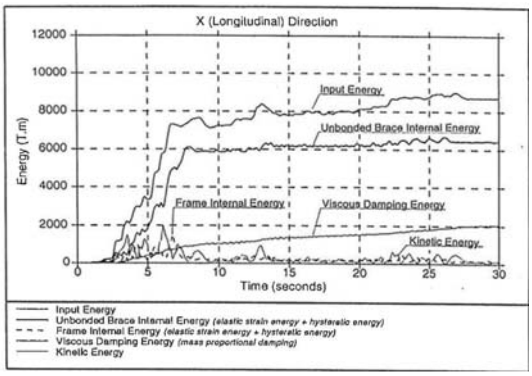

Figure 13: Seismic Energy Dissipation Plot

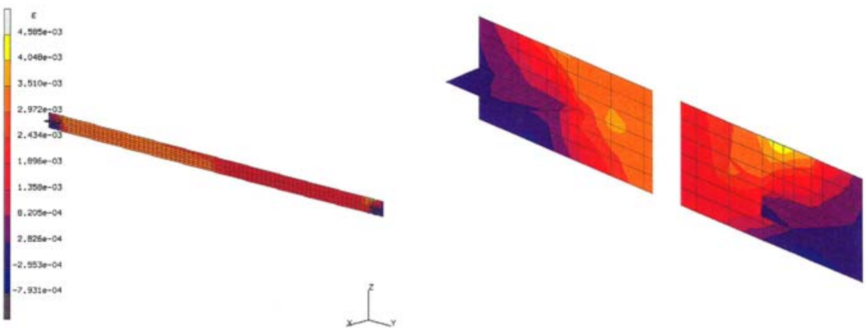
Figure 14: Unbonded Brace Analysis Result (elongation of 55 mm combined with self-weight)

6. ENSURING PROPER DUCTILITY AND FRACTURE TOUGHNESS

A new Japanese steel specification was utilized to control damage to the main superstructure and ensure ductile behavior of steel members without developing brittle fractures that had been encountered in Kobe. Fracture mechanics, the science of crack propagation, was used to assess the risk of brittle fracture. This project was the first case in which this was applied to seismic designs in Japan. 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)

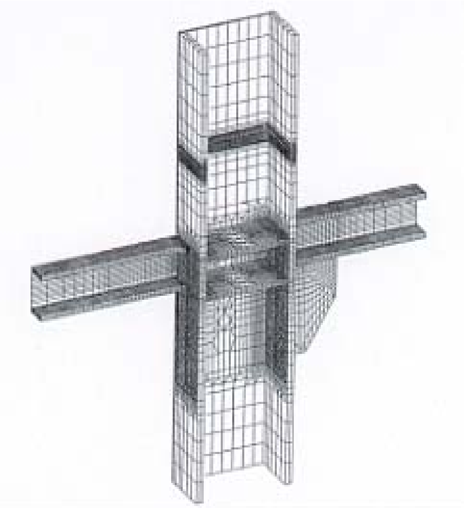

Figure 15: FEM Model of Column-Beam Joint

High strength steel, and optimal structural steel frame and connections details were applied to reduce the effect of stress concentrations. Specific details that were adopted includes:

- the use of round haunch detail at the beam flange/column flange connection
- removal of run on/run off tabs (which are usually left in place)
- prohibition of temporary attachments.

It has been confirmed that 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 and WES2805, which both describe methods for assessing the acceptability off 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, assumed flaw geometry was adopted. The stress condition for a typical supertruss column connection was established using a 3D non-linear finite element time history analysis and was also verified through full-scale experimentation. The model is shown in Figure 15, and stress contour results in Figures 16 and 17. 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.

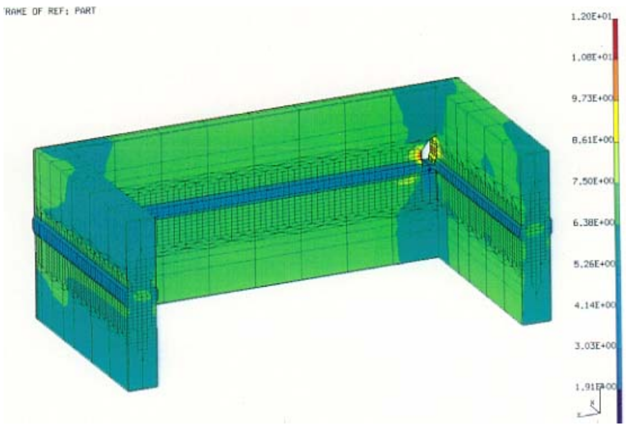

Figure 16: Column Welded Splice

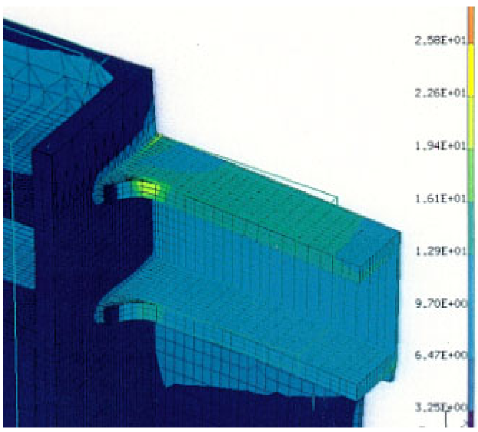

Figure 17: Column Beam Joint with round Haunch

8. CONCLUSION

The seismic engineering professionals in Japan still shaken by the wake of damage left behind by the 1995 Kobe earthquake, the design of the Osaka International Convention Center was undoubtedly carried out under special circumstances. Perhaps experiencing such pressures from the engineering community to further improve the seismic standard in Japan, along with architect Dr. Kurokawa's vision of a unique convention center that articulates the structural elements, the engineering design and analysis of this structure was vaulted into another level. Whilst over a decade has passed since the design stage, the intensity of the engineering techniques applied to bring about OICC to reality can still be consider advanced even by today's standards.

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