Use of Ultra-high Strength Steel in Building Frames Aiming at Major Countermeasure for Huge Earthquakes

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SUMMARY:
This paper aims to introduce the research results on design earthquake ground motions from an assumed near-field earthquake and on a seismic design method to prepare for it, reporting the ground motion characteristics and response characteristics recommended to be adopted for the design of buildings to be constructed in Osaka City. It shows that the use of ultra-high strength steel is more effective to assure structural safety against such pulse-wave ground motions as will cause a large story drift angle exceeding 1/50. Finally, the author's research subjects are briefed on the ultra-high strength steel of the order of tensile strength of 950 N/mm2 as used for building frames, illustrating an example of the building constructed with such steel.

Keywords: near-field earthquake ,pulse-like wave, enlarging deformation capacity, ultra-high strength steel

1. DESIGN EARTHQUAKE GROUND MOTIONS CAUSED BY ASSUMED "UEMACHI FAULT" EARTHQUAKE

1.1 Background of Design Earthquake Ground Motions

A pulse-like wave pattern of about one second or longer in period was observed near the seismic sources in more recent earthquakes, such as 1994 Northridge Earthquake and 1995 Hyogo-ken Nanbu Earthquake. Such pulse-wave ground motions are reported to give a large influence to buildings (Heaton, et al.,1995, Hall, et al.,1995).
In the Osaka City area, offices of Osaka Prefecture and Osaka City predicted the ground motions and publicize the earthquake damage assessment on the basis of their own research activities to provide against possible "Uemachi-fault" earthquake as near-field earthquake （Osaka pref. govt.,2007）.
Though the estimated ground motions vary in their intensity, they contain large estimated ground motions by far exceeding the design earthquake ground motions assumed in the building law. With such a background, the structural engineers from 50-odd design firms set up a study workshop in November 2009 in order to discuss how the design should respond to such large ground motions as possibly exceeding those assumed in the law. Under the guidance of professors in relevant disciplines residing in Osaka and its vicinity, the workshop worked out a seismic design guideline against the UEMACHI-fault earthquake after one and a half years of study (Taga,et al.,2011). This section of the paper outlines the research output of the workshop to which the author acted as a member.

1.2 Guiding Principles for Formulating Design Earthquake Ground Motions

To formulate the design earthquake ground motions, the workshop adopted the estimated ground motions prepared by Osaka City authorities in 2006. Their estimated ground motions consist of 500m-meshed estimated ground motions for assessment of earthquake damage due to a plural number of scenario earthquakes. In the case of UEMACHI-fault earthquake, it is assumed that the fault instantaneously moves in its entire length of 58 km as shown in Fig. 1. The 35 patterns of earthquake scenarios are formulated to predict the ground motions, according to uncertain elements of intensity of asperities, locations and starting point of rupture showing fault rupture patterns.

As shown in Fig. 2, since the formulated ground motions largely vary, the following three levels of design earthquake ground motions are set up. All of these levels exceed the level 2 defined in the Building Standard Law of Japan, i.e. “very scarcely occurring earthquake ground motions.” In actual practice, a structural engineer is to consult with the client to choose any one of those three.

Level 3A: Standard level when the UEMACHI-fault earthquake is taken into account and equivalent to the average level of the 35 scenarios of estimated ground motions.

Level 3B: Calling for a higher level of safety than for the standard level, and can cover a wider scope of variation in the ground motions, i.e. roughly 70% of the estimated ground motions of the 35 scenarios.

Level 3C: Calling for a much higher level of safety than for the standard level, and can cover a much wider scope of variation in the ground motions, i.e. 85% of the estimated ground motions of the 35 scenarios.

As shown in Fig. 3, Osaka City area is categorized into six zones according to the ground properties etc., mainly to the site amplification based on microtremor measurements and the distances from the fault. Such categorization is considered to duly reflect the geological property distribution of the near-surface layer and the depth distribution of the alluvial soil layer.

1.3 Design Earthquake Ground Motions in Horizontal Direction

When taking an overview of the estimated ground motions on ground surface, they can be grouped to those forming the comparatively flat response spectrum and those having strong pulse characteristics of very large response spectrum in which period characteristics are prominent. In view of this, two types of the ground motions, (1) flat type ground motion and (2) pulse type ground motions are used for design as input ground motions.

(1) Flat Type Ground Motion: simulated earthquake ground motions fitting to design response spectra which are set to have flat velocity response properties

(2) Pulse Type Ground Motion: pulse-like ground motions having strong predominant period properties, as selected from the estimated ground motions

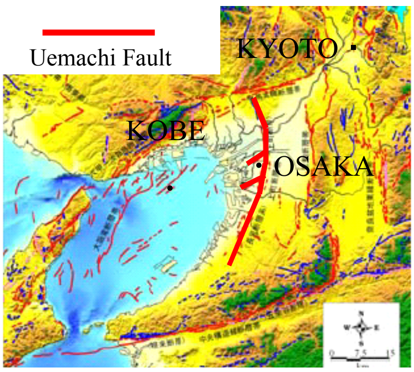


Figure 1. Assumed earthquake source fault

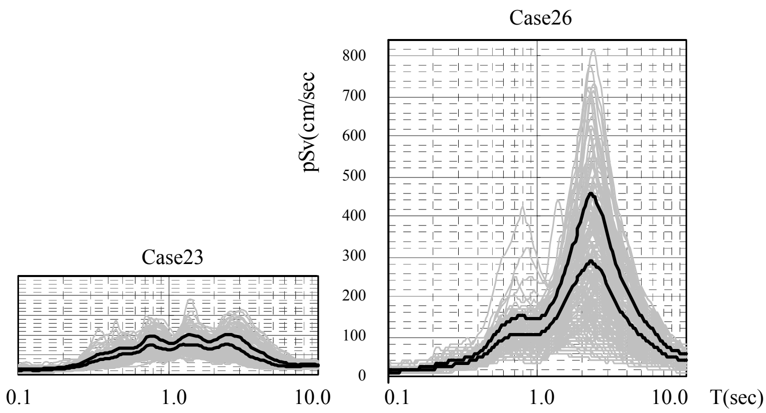


Figure 2. Variation of the formulated ground motions

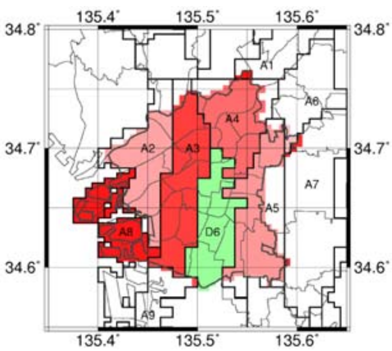


Figure 3. Zoning of Osaka City

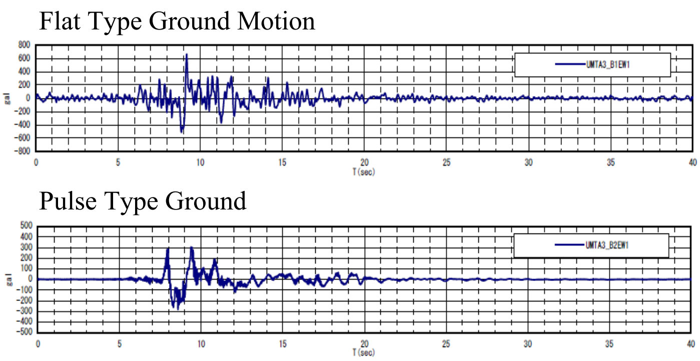


Figure 4. Example of input ground motion: ZoneA3 Level3B EW

These design earthquake ground motions are formulated to run in each zone at three different levels and in three motion patterns in each direction. One example of the input seismic ground motions is shown in Fig. 4.
Taking EW direction in Zone A3 where super-high-rise buildings are most populated, design response spectra pSv(h=5%) of Flat Type Ground Motions is shown in Fig. 5; pSv of Level 3B earthquake ground motion in Fig. 6; and pSv of Pulse Type Ground Motion (Level 3B) in Fig. 7. In particular, Fig. 5 concurrently shows the observed earthquake records of three types of motions (max. velocity at 50 cm/s) which are used as standard ground motions at Level 2 in the prevailing design approach.
When these three types of the input ground motions are compared with the code-elected Level 2 Ground Motions in a rough and ready way, 3A Level is 1.2 times larger, 3B Level 1.5 times larger and 3C Level 1.8 times larger.

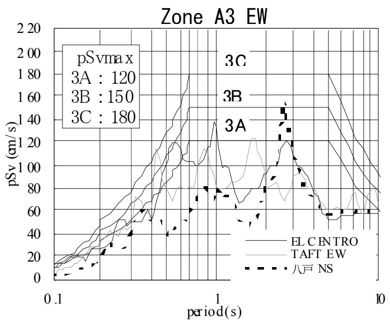


Figure 5. Design response spectrum for flat type ground motions

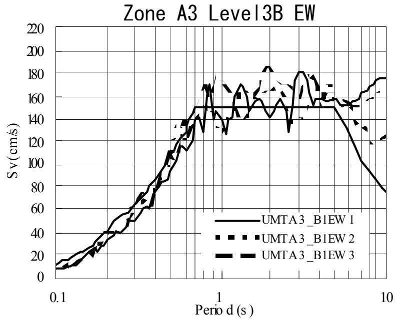


Figure 6. Sv of flat type ground motions(revel3B EW)

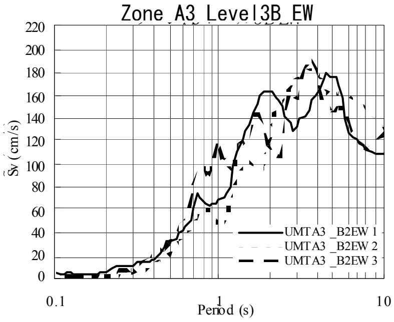
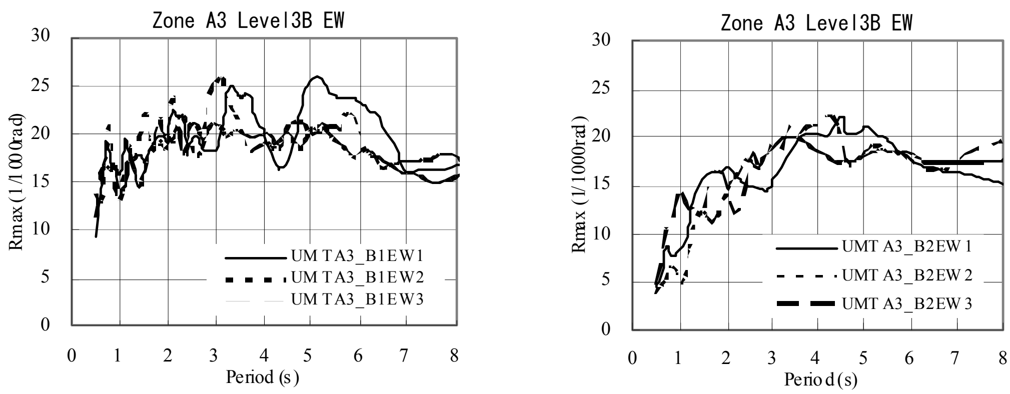


Figure 7. Sv of pulse type ground motions(revel3B EW)

The response deformation of a high-rise building is predicted with Rmax response spectra. This Rmax represents maximum story drift angle（elastic response）of a whole high-rise building as approximated with modal analysis, while assuming natural mode characteristics and vibration characteristics corresponding to the story drift angle of a building (Kamei,et al.,2010). Fig. 8 shows Rmax(h=2%) under the earthquake ground motions of Level 3B in EW direction in Zone A3. The horizontal axis represents fundamental natural period of a building. The figure confirms that both earthquake ground motions cause the maximum story drift angle of about 1/50 when assuming a high-rise building which would yield the story drift angle of the order of 1/100 under the currently used Level 2 earthquake ground motions. It is a major feature that a magnitude of this displacement response is less dependent on the changes in natural period or in stiffness.



flat type ground motions

pulse type ground motions

Figure 8. Example of Rmax response spectrum

In addition to the horizontal design earthquake ground motions above, sine wave pulse inputs also are proposed as effective study-purpose earthquake ground motions in an early stage of design or making comparative study of responses. Based on the empirical knowledge gained through observed earthquake records and estimated ground motions (Suzuki,et al.,2010), the sine wave pulse inputs are

used as study-purpose earthquake ground motions to be uniformly applied to the six zones in the forms of pulse amplitude of Vp = 100 to 150 cm/s and pulse period of Tp = 1, 2, 3.
The response spectrum of sine wave pulse input at Level 3B（Vp = 125cm/s）is shown in Fig. 9 and Rmax is shown in Fig. 10. The Rmax is about 1/50 to 1/60 similar to those of the Flat Type Ground Motions and the Pulse Type Motions.

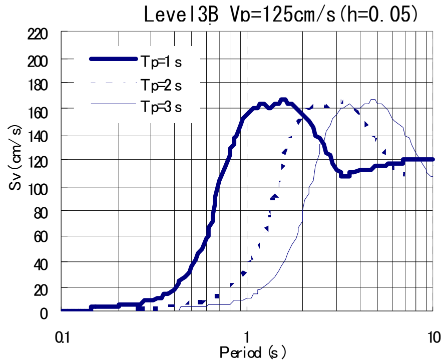


Figure 9. Sv of sine wave pulse input at Level 3B

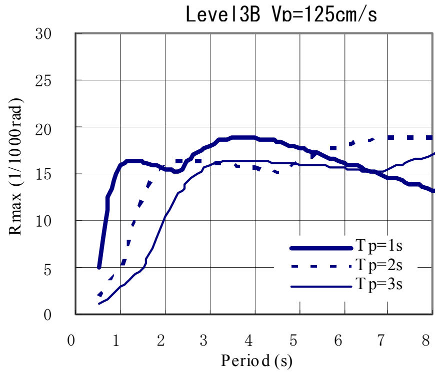


Figure 10. Rmax of sine wave pulse input at Level 3B

1.4 Target of Seismic Performance

Regarding the UEMACHI-fault earthquake, it is said that the occurrence probability within 30 years hence is two to three percent. In addition, the estimated ground motions largely vary from source to source; therefore, it is considered not large in the probability that new construction at respective sites would suffer from any catastrophic damage. As a consequence, it was determined that it would be permissible for designers to follow either of the following two conditions which are severer than such seismic required performance for high-rise buildings as codified and widely followed across Japan.
Limit State I: A building has allowances as compared with Limit State II which is considered to be ultimate state. If it withstands an earthquake in its mainshock, it would further withstand the subsequent aftershock and can be used for a certain period (subject to running repairs depending on the case).
Limit State II: Such a state as where a building is standing by itself, but on the verge of falling down. To strictly ascertain the state would require further analysis and investigation/research work.
As discussed above, according to the three levels of design earthquake ground motions and two design criteria, a structural engineer has to discuss with his/her client which combination of the former and the latter they choose. Fig. 11 illustrates the conceptual ideas of the relationship between the design earthquake ground motions and the design criteria.
Any combination covers the earthquake ground motions in excess of the codified boundary. Hence, the application of the combination has to be selected both by the client and the structural engineer; however, it is recommended for them to select “Limit State I or below for Level 3B”.

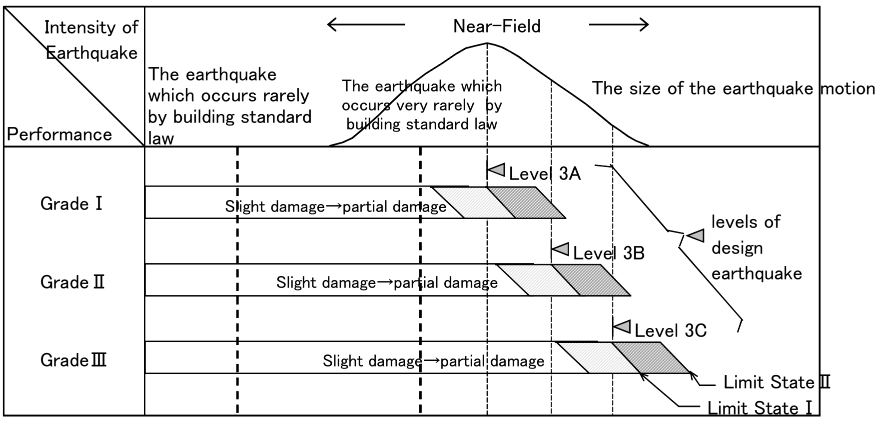


Figure 11. Relationship between the design earthquake ground motions and the design criteria

2. EFFECTIVE USE OF ULTRA-HIGH STRENGTH STEEL FOR ENLARGED DEFORMATION CAPACITY

2.1 Necessity of Enlarging Deformation Capacity and Use of Ultra-High Strength Steel

As described in Sub-section 1.3, the estimated ground motions of the UEMACHI-fault earthquake include the motion of which pulse period is at least in the order of one second and have large power. In the case of a building meeting the legislative boundary, if it is subjected to the recommended design earthquake ground motions, it would develop maximum displacement of two times larger than in the codified motions, sustaining serious damage. In order to strive for lessened damage against such motions, the basic design approach would be to enlarge elastic limit, while reserving plastic deformation capacity. Such an approach would be amplified as follows.
1)To enlarge the elastic limit to about 1.5 to 2 times, while keeping lateral stiffness at the current level, would require a material having large yield strength, for which purpose high strength steel would typically suffice. The reason for keeping lateral stiffness at the current level is twofold; firstly, it will evidently incur large cost to increase both stiffness and performance, though it is possible; and secondly, if the stiffness is lowered, the performance would be lowered against a small/medium earthquake or wind loading.
2) Since it cannot be negated for a building to sustain any larger earthquake ground motions than those estimated, steel members of ordinary grade should be used as beams expectable of plastic deformation capacity as part of beam failure mechanism.
3) By allowing for constructability and economy, any weld joint of high strength steel to same steel should be avoided.
When based on the foregoing reasoning, the target concept of restoring force characteristics can be shown as in Fig. 12. The section of a columns/beam to satisfy those requirements can be selected in the following procedures.
(1)Determine the rising ratio of bearing capacity to keep the target ductility ratio.
(2)Compute the beam section to meet the required bearing capacity (Any rising of stiffness should be restrained wherever possible).
(3)Determine the column section under the conditions that the stiffness of frame should be kept, while the conditions for bearing capacity are met.

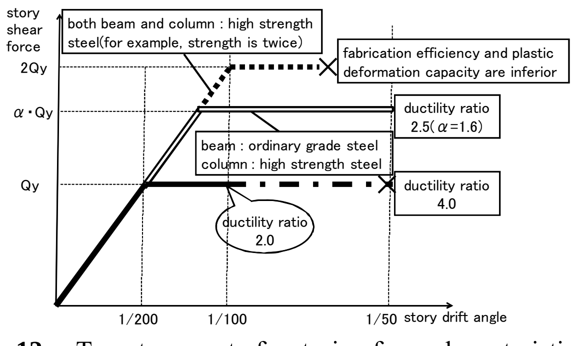
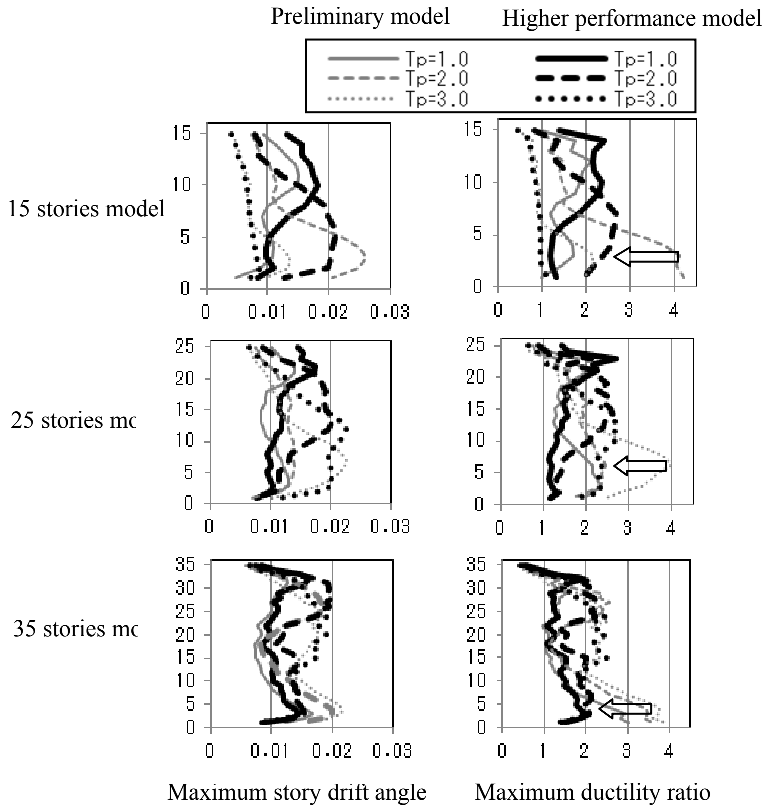


Figure 12. Target concept of restoring force characteristics

2.2 Effectiveness Verification of High Strength Steel with Trial Design Models

With an eye to verify the effectiveness of such high strength steel, an earthquake response was analyzed with a trial design model.
Preliminary models are designed in a way to meet the following conditions as prevailing seismic design criteria on the basis of model buildings of 15, 25 and 35 stories.
When the model is subjected to observed earthquake ground motions in which the peak ground velocity is standardized around 50 cm/s, the story drift angle should be of the order of 1/100 and the ductility ratio be not larger than 2.0.
When the sine wave pulse shown in Fig. 9 is inputted in the preliminary model, maximum story drift angle becomes in the order of 1/50 to 1/40 and maximum ductility ratio of a story about 4.
Next, the columns and beams of the story developing large response are changed in section size

according to Sub-section 2.1 so that maximum ductility ratio of the story will be adjusted to 2.5 or so (higher performance model).
The sine wave pulse is inputted to the higher performance model to check whether every story's ductility ratio meets the target ductility ratio. If any story exceeds the target, its sectional dimensions should be again reviewed, with same procedures being repeated.
The model's maximum response values as adjusted to become the target value or below are shown in Fig. 13 in conjunction with the results obtained through the preliminary model.
Thus, by utilizing the high strength steel according to the principle described in Sub-section 2.1, damage to frames due to the pulse-like ground motions can be dispersed and lessened.



Change of cross-sectional size

Preliminary model

model	Column(3^{rd} floor)	beam(3^{rd} floor
15	H-572×485×35×60	H-800×300×14×25
25	H-592×490×40×70	H-700×300×14×25
35	H-572×485×35×60	H-700×300×12×22

Higher performance mode

model	column(3^{rd} floor	beam(3^{rd} floor
15	H-542×493×39×$\boxed{39}$	H-900×300×19×32
25	H-567×515×36×$\boxed{39}$	H-900×300×19×34
35	H-527×479×38×$\boxed{38}$	H-800×300×14×25

※□：1000N steel,　all others are SN490B

Figure 13. Example of verification of high strength steel

3. PROPERTIES OF 1000 N CLASS ULTRA-HIGH STRENGTH STEEL FOR BUILDINGS

3.1 Background of Prevalence of High Strength Steel for buildings

High-strengthening demand for structural steel is now boosted as steel structures are scaled up in various sectors, not to speak of buildings. In the past, when the steel is high-strengthened, it, however, was prone to develop sensitivity of brittle fracture and weld crack, so that pre-heating and post-heating were indispensable in welding, or high level of interpass temperature control was needed. Then, the use of high strength steel of 780N/mm2 or over class was not so prevalent in reality.
To solve such problems, the author et al. started from 2003 to do research work for and succeeded in the use of ultra-high strength steel of tensile strength of 950N/mm2 class in building structure (Kawabata,et al.,2011,Sasaki,et al.,2011).

3.2 Mechanical Properties of 1000N Class Ultra-High Strength Steel

The chemical composition of the ultra-high strength steel developed as above is shown in Table 1; the results of tensile test and standard properties in Tables 2 and 3 respectively; stress-strain curve as compared with that of conventional 490N/mm2 class steel in Fig. 14. 1000N/mm2 class steel evidently has quite a large elastic range and, as compared with 490N/mm2 class steel, features small uniform elongation and a large yield ratio.

Table 4 shows the results of charpy impact testing and standard properties, from which it is known that it has extremely excellent fracture toughness at 0℃ as standard property temperature.

Table 1. Chemical compositions of SSS1000 [1/4t,mass%]

	Thickness	C	Si	Mn	P	S	Ceq	P_{CM}
Specification	6-50mm	≦ 0.16	≦ 0.55	≦ 1.50	≦ 0.015	≦ 0.008	≦ 0.62	≦ 0.34
Typical	50mm	0.10	0.19	0.97	0.005	<0.001	0.55	0.27

Ceq = C + Mn/6 + Si/24 + Ni/40 + Cr/5 + Mo/4 + V/14
P_{CM} = C + Si/30 + Mn/20 + Cu/20 + Ni/60 + Cr/20 + Mo/15 + V/10 + 5B

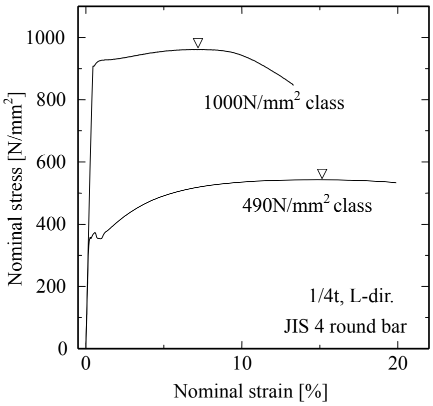


Figure 14. Stress-strain curve

Table 2. Tensile test results

Position	Dir.	Specimen configuration	Y.S. [N/mm²]	T.S. [N/mm²]	Y.R. [%]	El. [%]
1/4t	L	JIS Z2201 No.4 specimen	901	960	93.9	23.2
	C		909	966	94.2	22.8
1/2t	L		883	956	92.4	21.2
	C		892	962	92.7	20.5
Spec.	C	-	880/1060	950/1130	≦98	≧13

Table 3. Through thickness tensile test results

Dir.	Specimen configuration	T.S.[N/mm²]		R.A.[%]	
		Each	Av.	Each	Av.
Z	JIS G3199 Φ10	953	957	65.3	63.6
		960		64.0	
		959		61.6	
Spec.	-	-	-	≧15	≧25

Table 4. Charpy impact test results

Position	Dir.	Temp. [℃]	Specimen configuration	Absorbed energy [J]				vTrs [℃]
				Each			Av.	
1/4t	L	0	JIS Z2242 v-notch	266	256	259	260	-75
	C			275	287	276	279	-87
	Z			235	235	252	241	-
1/2t	L			237	229	236	234	-
	C			241	247	238	242	-
Spec.	L	0	-				≧70	

3.3 Performance of Welding Materials for 1000N Class Ultra-High Strength Steel for Buildings

To optimize the chemical composition for the welding material for high strength steel, certain development efforts were with an eye to improve strength and ductility, while avoiding low temperature cracking. For this purpose, welding materials were looked for subject to gas metal arc welding (hereinafter “GMAW”) and submerged-arc welding (“SAW”), both as welding methods used for building. U-groove weld cracking test was made in order to clarify preheating temperature for preventing the low temperature cracking from occurring in first layer welding, consequently the preheating temperature determined at 150℃.
Next, U-groove multipass weld cracking test was made to elucidate the relation of constraints to interpass temperature, postheating conditions and occurrence of traverse cracking. Lastly, it was ascertained that in both cases of welding materials for GMAW and SAW, no multipass weld cracking would be developed when the interpass temperature is lowered to 125℃, if postheating is applied. On the postheating conditions, it is considered that an equivalent effect might be obtained by appropriately combining several points of temperature and time elements.
In view of the mechanical properties, tension tests for deposited metal and charpy impact testing Test (1) were made under different heat input and interpass temperature conditions, with those test results evaluated in conjunction with U-groove multipass weld cracking test results. Finally, the application range of heat input and interpass temperature was clarified for each of GMAW and SAW welding materials. (Refer to the color-shaded parts in Figs. 15 and 16.)

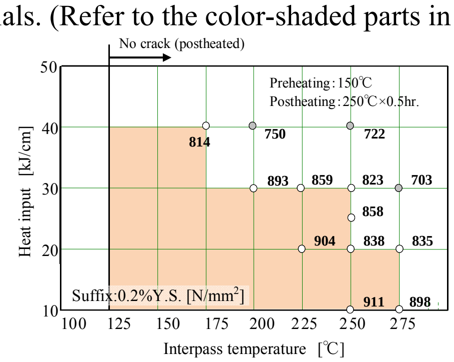


Figure 15. Crack preventing condition（GMAW）

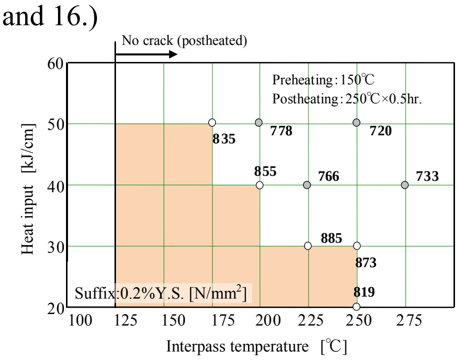


Figure 16. Crack preventing condition（SAW）

4. EXAMPLE OF A BUILDING INCORPORATING 1000N CLASS ULTRA-HIGH STRENGTH STEEL FOR BUILDINGS

4.1 Outlines of the Building and Structural Planning

Here, an example of the building for which 1000N class ultra-high strength steel referred to in Section 3 is used, is introduced. Figs. 17 and 18 show framing plan and framing elevation of the building. It is a research-related building of steel construction, six stories above ground, 135m x 35m in building plan, and about 30m in height.

In the building plan, 1st floor is provided with entrance zone, and 2nd floors and above with office zones. Structural system is fitted to respective spatial zones. On the typical floor, a column-free huge space is provided in 133m x 23m to assure flexibility in floor plan arrangement and is supported by 23m-long girders and exposed, double-skin type of steel members.

Columns and beams are basically composed of welded H-sections of SN490 or SN400N/mm2 class steel. Their structural stiffness is maintained with two-tier beams and pair columns, paying due thought to member's proportions as a design element.

On the 1st floor, columns are made of 1000N class ultra-high strength steel, with buckling-restrained braces provided in a balancing way to act as vibration control system which enables almost all seismic energy to be absorbed in a intensive way, and yet the columns and beams are designed to be kept below the elastic limit at Level 2 earthquake.

Not to speak of Level 2 earthquake ground motions, secondary fixing members are designed to yield in advance even when hit by twice larger ground motions, compared with Level 2, so that 1000N class steel members can be suppressed below the allowable stress.

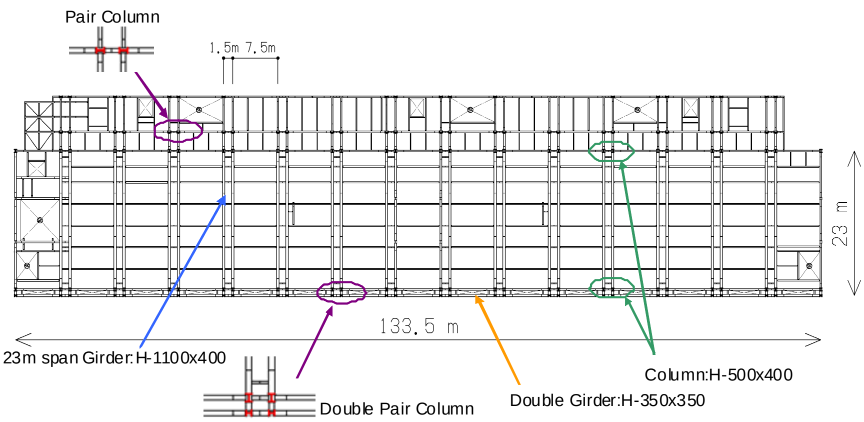
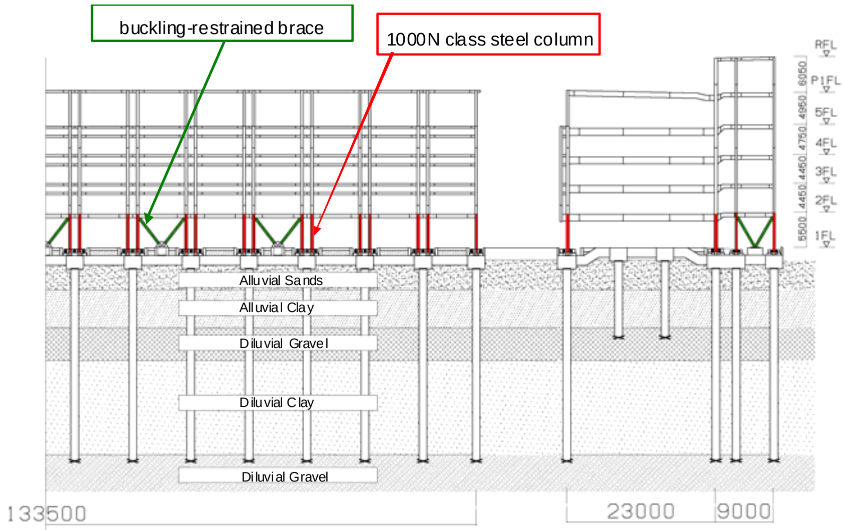


Figure 17. Framing plan of the building



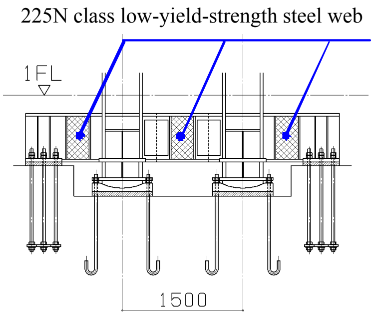


Figure 18. Framing elevation of the building

Figure 19. Detail of column bases

4.2 Outline of Seismic Design

Fig. 19 shows the detailed column bases for 1000N class steel columns. The column bases are pin-supported. A short bracket to resist bending force acting on column is provided with a trigger mechanism at the web of bracket and the trigger mechanism's 225N class low-yield-strength steel is designed to yield in advance of 1000N class steel column.
The seismic safety is verified through time history response analysis. The response spectrum of the ground motions used for response analysis is shown in Fig. 20. This figure shows the response spectrum in engineering basis taken as 1.00 to 2.00 times larger than in Level 2 ground motions. The building's fundamental natural period is about 0.8 seconds in terms of initial stiffness, about 0.9 seconds in equivalent period at maximum deformation under Level 2 ground motions and about 1.0 second in equivalent period under two-times larger input than in Level 2 ground motions.
Fig. 21 shows the results of load-deformation relationship and time history response analysis for the 1st and 2nd floors in X direction. Where ultimate horizontal resistant force is defined as story shear force when story drift angle of the 1st floor becomes 1/70, it is roughly equal to maximum response story shear force under two-times larger input than in Level 2. Fig. 22 shows one example of stress conditions in 45 degree angle direction under two-times larger input than in Level 2. Even in the case of input in 45 degree direction, 1000N class steel columns are kept under the allowable stress or elastic limit.
This building stands in Amagasaki City westerly abutting to Osaka City. Though it is considered less influenced by the UEMACHI-fault earthquake, it was subjected to the analysis using sine wave pulse input to be on the safe side. The response result is shown in Fig. 21. The response under Level 3B input is roughly equal to that under two-times larger input than in Level 2 ground motions and the response is well kept at Limit State I against the pulse-wave ground motions at Level 3B.

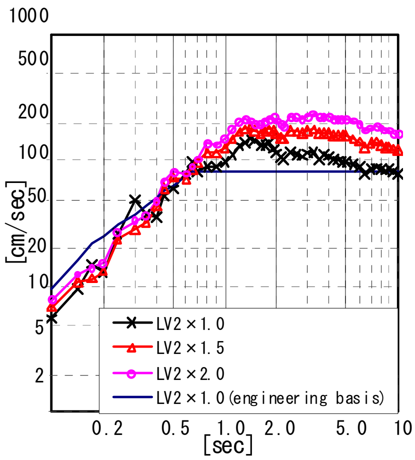


Figure 20. The response spectrum of the ground motions used for response analysis

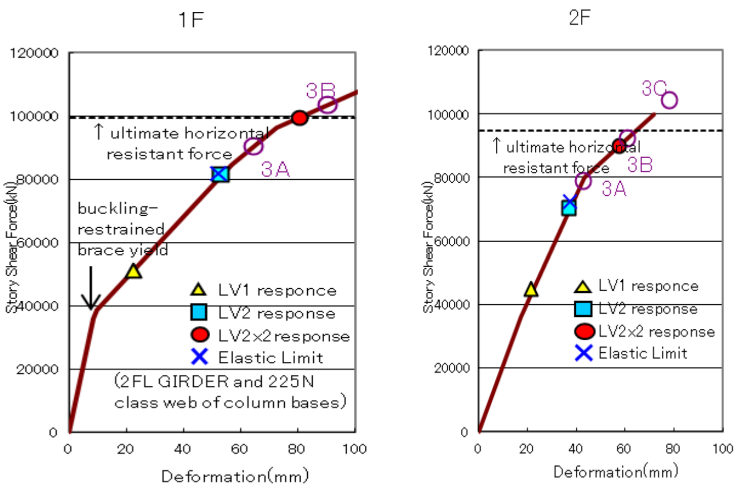


Figure 21. The results of load-deformation relationship and time history response analysis

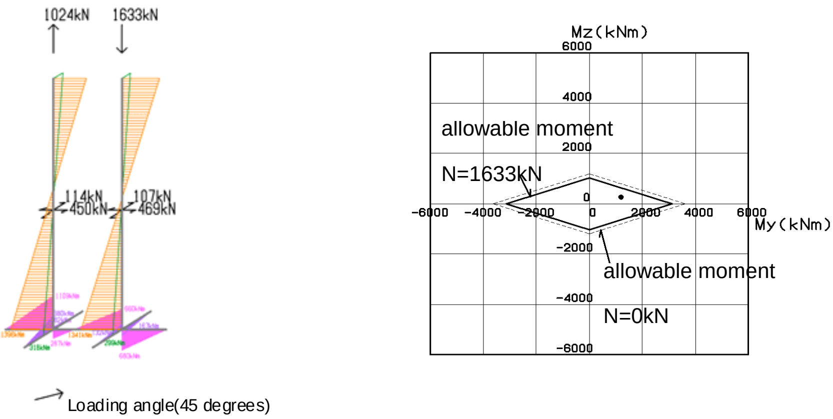


Figure 22. Example of stress conditions in 45 degree angle direction under two-times larger input than in Level 2

4.3 Studies toward Actual Construction

In addition to those tests made in the course of development of the ultra-high strength steel, cyclic loading tests were made at column-beam joints adopted in the actual building eventually to find that when the welding conditions and weld details for 490N/mm2 class steel were applied, 1000N/mm2 class steel portion did not show noticeable softening. Thus, it was verified that the 490 N/mm2 steel beams did exert the plastic deformation capacity.
Also, prior to the field fabrication of welded H-columns of 1000N class steel, pilot fabrication tests were done to ascertain the practicability of sound welding operation, while securing proper fabrication efficiency.
After clearing all these analytical studies, the construction of the building was started in June 2010 and completed in October 2011.

5. CONCLUSION

The Osaka City, one of the megacities across Japan, has to provide against the UEMACHI-fault earthquake as a near-field earthquake. To cope with that earthquake, the author has shown the design earthquake ground motions and the associated fundamental design approach for structural design.
Those design earthquake ground motions give the pulse period of the order of about one second or longer and heavy power enough to cause large displacement response in buildings. To mitigate catastrophic damage to buildings, this paper has showed it very effective to keep large the elastic limit by the use of ultra-high strength steel.
Also it has showed an example of the building in which sufficiently large seismic performance is enabled by actually using the ultra-high strength steel having tensile strength of 950N/mm2 class.
The author hopes for such high strength steel being put into more practical use by virtue of improved properties.

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