

[Document Type] Specification

[Title of the Invention] BEAM-TO-COLUMN CONNECTION

[Technical Field of the Invention]

[0001]

The present invention relates to a beam-to-column connection.

Priority is claimed on Japanese Patent Application No. 2016-137010, filed on July 11, 2016, the content of which is incorporated herein by reference.

[Related Art]

[0002]

In the related art, a building (steel framed building) employs a beam-to-column connection (for example, refer to Patent Document 1).

The beam-to-column connection of Patent Document 1 includes a non-diaphragm steel tube column and a beam directly connected to this steel tube column. The steel tube column is constituted of a column-beam connection portion to which the beam is directly welded, and a non-column-beam connection portion to which the beam is not connected. The outer diameter of the column-beam connection portion and the outer diameter of the non-column-beam connection portion are the same as each other. The thickness of the column-beam connection portion is greater than the thickness of the non-column-beam connection portion.

In a non-diaphragm beam-to-column connection, the number of components constituting the beam-to-column connection is reduced. Moreover, since the number of steps of machining the components is reduced, the manufacturing cost of the beam-to-column connection is suppressed.

[Prior Art Document]

[Patent Document]

[0003]

[Patent Document 1] Japanese Unexamined Patent Application, First
Publication No.2010-229660

[Disclosure of the Invention]

[Problems to be Solved by the Invention]

[0004]

Here, in place of a steel tube column in Patent Document 1, there are cases of using a steel column having an H-shaped cross section, a flanged cruciform cross section, or the like. Then, for example, in a case where a flange of the steel column has a non-standard plate thickness, the steel column is manufactured by welding a web and the flange to each other, instead of rolling. In this case, a beam is joined to the flange of the steel column by welding.

[0005]

If a load acts on the beam, bending moment acts on a beam-to-column connection. Due to this bending moment, a pulled portion and a compressed portion are generated in the flange of the steel column. In this case, if the web and the flange of the steel column are joined to each other by partial joint penetration welding or fillet welding, a notched incomplete penetration, in which the web and the flange are not joined to each other, is present in a joint portion therebetween. Therefore, there is concern that the joint portion will break due to the load acting on the beam, while having this incomplete penetration as an origin.

[0006]

In order to prevent an incomplete penetration from being formed in the joint portion between the web and the flange, the web and the flange have been entirely joined to each other by complete joint penetration welding to realize full strength

joining between the web and the flange. Due to this full strength joining, a crack is unlikely to be generated in the joint portion.

However, in a case where a web and a flange are joined to each other by complete joint penetration welding, manufacturing cost increases compared to a case where a web and a flange are joined to each other by partial joint penetration welding or fillet welding. The reason is that a part in which a web is butting-welded to a flange requires groove cutting in complete joint penetration welding, and more weld metals are required to perform welding for back-filling of a space generated due to the groove cutting. As another reason, in complete joint penetration welding, there is a need to check that an incomplete penetration is not formed, by performing an ultrasonic flaw detecting test or the like after welding.

There is concern that such an increase in manufacturing cost due to the complete joint penetration welding will offset reduction of the manufacturing cost obtained by adopting the non-diaphragm type, so that an advantage of the non-diaphragm type will be impaired.

[0007]

The present invention has been made in consideration of the foregoing circumstances, and an object thereof is to provide a beam-to-column connection in which breaking of a joint portion between a web and a flange of a steel column is restrained when a load acts on a beam, and an increase in manufacturing cost can be restrained.

[Means for Solving the Problem]

[0008]

In order to solve the foregoing problems, the present invention employs the following aspects.

(1) According to an aspect of the present invention, there is provided a beam-to-column connection which is a structure of a non-diaphragm type. The beam-to-column connection includes a column that has a steel reinforced concrete structure or a steel framed structure, and a beam that has a steel framed structure joined to the column. The column has a steel column which has a flanged cruciform cross section or an H-shaped cross section and includes a web, a flange, and a full strength joint portion and a non-full strength joint portion joining the web and the flange to each other. The full strength joint portion is formed in at least a range in which tensile stress acts on the flange of the steel column when bending moment acts on the beam.

[0009]

(2) In the aspect of (1), the full strength joint portion may be formed in a range of a value equal to or greater than $H_b + 2y$ obtained by using the following expression, while having a center of a depth of the beam as a center of the range. Here, in the following expression, B_c is a width (mm) of the flange of the steel column, t_{cf} is a plate thickness (mm) of the flange of the steel column, t_{cw} is a plate thickness (mm) of the web of the steel column, σ_{cfy} is yield strength (N/mm²) of the flange of the steel column, σ_{cwy} is yield strength (N/mm²) of the web of the steel column, H_b is a depth (mm) of the beam, F_c is compression strength (N/mm²) of concrete of the column, and d is a cover thickness (mm) of concrete with respect to the flange of the steel column. F_c and d are zero when the column has the steel framed structure.

[0010]

[Math 1]

$$y = B_c \sqrt{\frac{\sigma_{cfy}}{\sigma_{cwy}}} \cdot \sqrt{\frac{t_{cf}/t_{cw}}{2(B_c/2t_{cf})}} \cdot \sqrt{\frac{1}{1 + 1.24d \sqrt{F_c/(\sigma_{cwy}t_{cw})}}}$$

[Effects of the Invention]

[0011]

According to the aspect of (1) described above, the full strength joint portion joining the flange and the web of the steel column to each other is formed in the range in which tensile stress acts on the flange of the steel column. Therefore, an incomplete penetration of the flange and the web is not formed in this range. In a steel column, a joint portion between the flange and the web is likely to break in the range in which tensile stress acts on the flange while having the incomplete penetration as an origin. However, since the incomplete penetration is not formed in this range, the joint portion is unlikely to break when a load acts on the beam. In addition, the non-full strength joint portion is formed in a range other than the range in which the full strength joint portion is formed. Accordingly, breaking of the joint portion between the flange and the web in the steel column is restrained and an increase in manufacturing cost can be restrained.

[0012]

According to the aspect of (2), the full strength joint portion is formed in a range pulled due to the load acting on the beam from above and below (range of the value of H_b+2y having the center of the depth of the beam as the center of the range). In addition, the non-full strength joint portion is formed in a range other than the range in which the full strength joint portion is formed. Therefore, even if a load acts on the beam from above and below, breaking of the joint portion between the flange and the web in the steel column can be restrained, and an increase in manufacturing cost can be restrained.

[Brief Description of the Drawings]

[0013]

FIG. 1 is a perspective view showing a beam-to-column connection according

to a first embodiment of the present invention, and is a view thereof partially seen through.

FIG. 2 is a cross-sectional view showing a column of the beam-to-column connection.

FIG. 3 is a partial cross-sectional view showing a non-full strength joint portion in a steel column of the column.

FIG. 4 is a partial cross-sectional view showing another non-full strength joint portion in the steel column of the column.

FIG. 5 is a cross-sectional view showing the beam-to-column connection and is a view showing a collapse mechanism (of a joint) at the time of full plastic strength.

FIG. 6 is a side view seen along line A1-A1 in FIG. 5 and is a view showing cone failure of cover concrete.

FIG. 7 is a side view seen along line A1-A1 in FIG. 5 and is a view showing bearing failure of concrete.

FIG. 8 is a view showing actual stress-strain characteristics of a steel.

FIG. 9 is a view showing modeled stress-strain characteristics of a steel.

FIG. 10 is a side view of the steel column and is a view showing a range in which a full strength joint portion and the non-full strength joint portion are formed.

FIG. 11 is a view for verifying accuracy in estimating a variable y in a second embodiment of the present invention.

FIG. 12 is a perspective view showing an analytic model used in analysis of a beam-to-column connection in a third embodiment of the present invention.

FIG. 13 is a perspective view showing a main portion of the analytic model.

FIG. 14 is a view seen in a direction A in FIG. 13.

FIG. 15 is a view showing stress-strain characteristics used for elements of a

steel column and a beam in finite element analysis of the beam-to-column connection.

FIG. 16 is a view showing stress-strain characteristics used for an element of a weld metal portion in the finite element analysis of the beam-to-column connection.

FIG. 17 is a view showing an example of a result of plastic strain obtained from a result of the finite element analysis of the beam-to-column connection.

FIG. 18 is a view showing a relationship between a flange width of the beam and a distribution range of the plastic strain in a state where an end portion of the beam is rotated by 0.02 (radian).

FIG. 19 is a view showing the relationship between the flange width of the beam and the distribution range of the plastic strain in a state where the end portion of the beam is rotated by 0.03 (radian).

FIG. 20 is a view showing the relationship between the flange width of the beam and the distribution range of the plastic strain in a state where the end portion of the beam is rotated by 0.04 (radian).

[Embodiments of the Invention]

[0014]

Hereinafter, each of embodiments of the present invention will be described in detail with reference to the drawings. In this specification and the drawings, the same reference signs are applied to constituent elements having substantially the same functional configuration, and duplicated description thereof will be omitted.

[0015]

(First embodiment)

FIG. 1 is a perspective view showing a beam-to-column connection 1 according to a first embodiment of the present invention. In FIG. 1, in order to make the invention easy to understand, the beam-to-column connection 1 is displayed in a

manner of being partially seen through. As shown in FIG. 1, the beam-to-column connection 1 includes a column 11 having a steel reinforced concrete (SRC) structure, a beam 21 having a steel framed structure, and a joint portion 31 joining the column 11 and the beam 21 to each other. The beam-to-column connection 1 is a structure of a type in which no diaphragm (stiffener) is used when the beam 21 is joined to the column 11, that is, a structure of a non-diaphragm type.

[0016]

The column 11 has a steel reinforced concrete structure as described above and includes a steel column 12 having a web 14 and a flange 13, a plurality of reinforcing rods 15 surrounding the steel column 12, and concrete 16. The steel column 12 and the plurality of reinforcing rods 15 are embedded inside the concrete 16. The column 11 is not limited to the steel reinforced concrete structure and may be a column having a steel framed structure not including the reinforcing rods 15 and the concrete 16, for example.

[0017]

FIG. 2 is a view showing the steel column 12 of the beam-to-column connection 1 and is a cross-sectional view thereof in a case of being seen in a cross section perpendicular to a member length direction (longitudinal direction) and passing through a full strength joint portion 17. In addition, FIG. 3 is a view showing the steel column 12 of the beam-to-column connection 1 and is a partial cross-sectional view thereof in a case of being seen in a cross section perpendicular to the member length direction and passing through a non-full strength joint portion 18A. In addition, FIG. 4 is a view showing the steel column 12 of the beam-to-column connection 1 and is a partial cross-sectional view thereof in a case of being seen in a cross section perpendicular to the member length direction and passing through a non-full strength

joint portion 18B.

[0018]

As shown in FIGS. 1 to 4, the steel column 12 having a flanged cruciform cross section is obtained by welding and joining an end portion of the web 14 (end portion of the web 14 in a width direction) and the flange 13 to each other. The end portion of the web 14 and the flange 13 are joined to each other by the full strength joint portion 17 formed by complete joint penetration welding (part subjected to the full strength joining) as shown in FIG. 2, the non-full strength joint portion 18A formed by partial joint penetration welding (part subjected to joining by a method other than full strength joining) as shown in FIG. 3, and the non-full strength joint portion 18B formed by fillet welding (part subjected to joining by a method other than full strength joining) as shown in FIG. 4.

The end portion of the web 14 and the flange 13 may be joined to each other by the full strength joint portion 17 and the non-full strength joint portion 18A or may be joined to each other by the full strength joint portion 17 and the non-full strength joint portion 18B. That is, the end portion of the web 14 and the flange 13 may be joined to each other by the full strength joint portion 17 and any one of the non-full strength joint portion 18A and the non-full strength joint portion 18B.

[0019]

Here, “complete joint penetration welding” denotes that the entire cross section of a base material (in the present invention, a web of a steel column) is completely welded by providing a groove in a weld joint portion, and a full strength joint portion (a joint portion having proof stress which does not fall below the proof stress of the base material) in which a weld throat thickness becomes equal to or greater than the thickness (plate thickness) of the base material is formed. In addition, “partial joint

penetration welding” is a welding method in which a groove is partially provided in the weld joint portion and a part of a cross section of the base material is welded.

Examples thereof include deformed fillet welding. In addition, in “fillet welding”, a weld metal having a triangular cross section is mounted in a corner between the web and the flange of the steel column which are orthogonal to each other and are installed without providing a groove.

[0020]

Partial joint penetration welding and fillet welding are welding methods allowing the presence of a non-welding deposition portion within a cross section of a welding joint. In addition, “Standard Specification of Construction Work of Architectural Institute of Japan, and Commentary on the same, JASS6, Steel Frame Construction” published by the Architectural Institute of Japan discloses that a full strength joint portion formed by complete joint penetration welding has to be checked for having no internal defect by performing an ultrasonic flaw detecting test or the like. Therefore, even in a case where the entire cross section butting the base material in accordance with the welding specification is completely welded, it is desirable to check that there is no internal defect by performing an ultrasonic flaw detecting test or the like after welding.

[0021]

That is, a part in which the flange 13 and the web 14 of the steel column 12 are subjected to the full strength joining is the full strength joint portion 17 in which the flange 13 and the web 14 of the steel column 12 are joined to each other by complete joint penetration welding. On the other hand, a part in which the flange 13 and the web 14 of the steel column 12 are subjected to joining by a method other than full strength joining is the non-full strength joint portion 18A in which the flange 13 and the

web 14 of the steel column 12 are joined to each other by partial joint penetration welding (non-full strength joining), or the non-full strength joint portion 18B in which the flange 13 and the web 14 of the steel column 12 are joined to each other by fillet welding (non-full strength joining).

[0022]

For example, the non-full strength joint portion 18A formed by partial joint penetration welding as shown in FIG. 3 is constituted of weld metal portions 18a and 18b which are formed in the end portion of the web 14 by submerged arc welding (SAW) and join the web 14 and the flange 13 to each other. These weld metal portions 18a and 18b are formed to have the web 14 interposed therebetween in the thickness direction (plate thickness direction) thereof. As described above, since the non-full strength joint portion 18A is formed by partial joint penetration welding, a notched incomplete penetration 18c is formed between the weld metal portion 18a and the weld metal portion 18b.

[0023]

Similar to the non-full strength joint portion 18A, the non-full strength joint portion 18B formed by fillet welding as shown in FIG. 4 is formed in the end portion of the web 14 and is constituted of weld metal portions 18d and 18e joining the web 14 and the flange 13 to each other. Then, since the non-full strength joint portion 18B is formed by fillet welding as described above, a notched incomplete penetration 18f is formed between the weld metal portion 18d and the weld metal portion 18e.

In this specification, the non-full strength joint portion 18A and the non-full strength joint portion 18B are generically named as a non-full strength joint portion 18 when being referred to without being distinguished from each other.

[0024]

On the other hand, in the full strength joint portion 17 formed by complete joint penetration welding, a pair of weld metal portions is integrated without having an incomplete penetration formed therebetween (refer to FIG. 2).

[0025]

In the steel column 12 of the beam-to-column connection 1, the web 14 and the flange 13 to which the beam 21 is joined are subjected to the full strength joining in a predetermined range in the member length direction (longitudinal direction) of the steel column 12 and are joined to each other by a method other than full strength joining in a range other than the predetermined range. In other words, in the steel column 12 of the beam-to-column connection 1, the full strength joint portion 17 joining the flange 13 and the web 14 to each other is formed in the predetermined range, and the non-full strength joint portion 18 joining the flange 13 and the web 14 to each other is formed in a range other than the predetermined range (refer to FIG. 10). That is, in the steel column 12, in a case of being seen along the member length direction (longitudinal direction) thereof, the full strength joint portion 17 and the non-full strength joint portion 18 joining the flange 13 and the web 14 to each other are formed, and no gap is generated between the full strength joint portion 17 and the non-full strength joint portion 18 (the full strength joint portion 17 and the non-full strength joint portion 18 are continuously formed).

The predetermined range will be described in detail below. In addition, the steel column 12 is not limited to that having a flanged cruciform cross section as described above and may have an H-shaped cross section in which the end portion of the web 14 and the flange 13 are welded to each other.

[0026]

The plurality of reinforcing rods 15 is disposed on a rectangular reference line

in a case of being seen in the member length direction of the steel column 12 as shown in FIG. 2.

In the concrete 16, a cross section perpendicular to the member length direction has a square shape or a rectangular shape.

[0027]

As shown in FIG. 1, the beam 21 is made of a steel H-shape and has a web 24 and a pair of flanges 23 joined to both end portions of the web 24. The beam 21 is joined to an outer surface of the flange 13 of the steel column 12 and extends in a manner perpendicular to the flange 13. The beam 21 is a steel frame and is constituted of a steel sheet, for example.

[0028]

The flanges 23 of the beam 21 are joined to the flange 13 of the steel column 12 in the joint portion 31 by welding. The web 24 of the beam 21 is joined to the flange 13 of the steel column 12 by welding or high strength bolt friction joining via a shear plate. The joint portion 31 in which the flange 13 of the steel column 12 and the beam 21 are joined to each other (including a weld, and the steel column 12 and the beam 21 in the vicinity of the weld) is surrounded by the concrete 16. That is, a part of the beam 21 is embedded in the concrete 16 of the steel column 12.

[0029]

In this specification, the web 14 and the flange 13 of the steel column 12 are joined to each other on the premise of weld joining. However, for example, adhesive joining, pressure joining, or amorphous joining may be used without being limited to weld joining. That is, the full strength joint portion 17 joining the web 14 and the flange 13 may be formed by adhesive joining, pressure joining, amorphous joining, or the like. Similarly, the non-full strength joint portion 18 joining the web 14 and the

flange 13 may be formed by adhesive joining, pressure joining, amorphous joining, or the like.

[0030]

[1. Collapse Mechanism (of a joint) Proposed in Present Invention]

Next, a collapse mechanism (of a joint) of the beam-to-column connection 1 will be described. In the present invention, a mechanism shown in FIGS. 5 to 7 is assumed. FIGS. 5 to 7 show a state after a load acts from above the beam 21 (a load acts from above on the flange 23 positioned on an upper side between the pair of flanges 23 of the beam 21) and the beam-to-column connection 1 is deformed due to the bending moment generated in the beam-to-column connection 1 due to this load. FIG. 5 is a partial cross-sectional view in a case of being seen in a cross section perpendicular to the flanges 23 of the beam 21 and is a view showing the collapse mechanism (of a joint) at the time of full plastic strength. In addition, FIG. 6 is a side view seen along A1-A1 in FIG. 5 and is a view showing cone failure of cover concrete. In addition, FIG. 7 is a side view seen along line A1-A1 in FIG. 5 and is a view showing the bearing failure of concrete.

[0031]

As shown in FIG. 5, it is assumed that plastic hinges 13₁ and 13₃ are formed above the flange 13 of the steel column 12, and plastic hinges 13₂ and 13₄ are formed below thereof. That is, in parts to which the pair of flanges 23 of the beam 21 is connected in the flange 13 to which the beam 21 is joined, the pairs of plastic hinges 13₁ and 13₃, and the pairs of hinges 13₂ and 13₄ are formed respectively. It is assumed that plastic rotation angles θ_1 and θ_2 (which will be described below) are deformed from the state of zero (radian) to a positive state of the plastic rotation angles θ_1 and θ_2 as shown in FIGS. 5 to 7.

With respect to a force of the flange 23 on the tension side of the beam 21 (in this example, a flange 23₁ of the pair of flanges 23 of the beam 21 positioned on the upper side: which will hereinafter be referred to as an upper flange 23₁) drawing the flange 13 of the steel column 12 out of the plane, the steel column 12 (steel frame) causes out-of-plane deformation of the flange 13 and a local yield of the web 14. Accordingly, a local yield 14a is formed in the web 14 of the steel column 12. In addition, the plastic hinge 13₁ is formed in the flange 13 of the steel column 12.

When the flange 13 of the steel column 12 is deformed out of the plane, cone failure is caused in cover concrete 16a on the outer side of the flange 13. A side surface 16a1 of the cover concrete 16a is shown in FIG. 6 by applying hatching thereto.

[0032]

As shown in FIG. 5, with respect to a force of the flange 23 on the compression side of the beam 21 (in this example, a flange 23₂ of the pair of flanges 23 of the beam 21 positioned on the lower side: which will hereinafter be referred to as a lower flange 23₂) pushing the flange 13 of the steel column 12 out of the plane, it is assumed that the local yield of the web 14 is generated due to the flange 13 deformed to the inner side out of the plane, thereby resulting in bearing failure of the concrete 16 on the inner side of the flange 13. That is, a bearing failure portion 16b is formed in the concrete 16. The bearing failure portion 16b is shown in FIG. 7 by applying hatching thereto. In addition, the plastic hinge 13₂ is formed in the flange 13 of the steel column 12.

Generally, bearing strength of concrete becomes greater than cone failure strength. Therefore, a neutral axis C1 satisfying a condition of equilibrium within a cross section of the joint portion 31 is positioned on the flange 23₂ side on the compression side of the center of a beam depth H_b (depth of a beam) (mm) in an up-down direction.

[0033]

In this the collapse mechanism (of a joint), a state where the joint portion 31 is rotated by an angle θ (radian) with respect to the bending moment of the end portion of the beam 21 is assumed. However, the angle θ is a very small angle and can be approximated as $\tan \theta = \theta$ or the like.

Variables used in this collapse mechanism (of a joint) are x , y , and z (mm) as shown in the diagram. The variable x is a coefficient determining the position of the neutral axis C1 with respect to the bending moment of the end portion of the beam 21, and can have any value ranging from 0 to 1. Specifically, the variable x is a ratio of the distance from the outer surface of the flange 23₁ on the tension side of the beam 21 to the neutral axis C1 to the beam depth H_b when bending moment acts on the end portion of the beam 21. In addition, the variable y is a length between the plastic hinge 13₁ and the plastic hinge 13₃ positioned above the upper flange 23₁ of the beam 21 in the member length direction of the steel column 12. In addition, the variable z is a length between the plastic hinge 13₂ and the plastic hinge 13₄ positioned below the lower flange 23₂ of the beam 21 in the member length direction of the steel column 12. The variables y and z can have any positive number (value greater than zero).

With these variables x , y , and z , out-of-plane deformation amounts δ_1 and δ_2 (mm) of intersecting portions between the flange 13 of the steel column 12, and the flanges 23₁ and 23₂ of the beam 21 can be expressed by the following Expressions (3) and (4) using the following Expressions (1) and (2). Here, the plate thickness of the flange 23 of the beam 21 is referred to as t_{bf} (mm), and the width of a rigid zone assumed in the intersecting portions between the flange 13 and the flanges 23 is referred to as t' (mm).

[0034]

[Math 2]

$$t' = t_{bf} \quad \cdot \quad (1)$$

$$H = H_b + t' - t_{bf} \quad \cdot \quad (2)$$

$$\delta_1 = x\theta H \quad \cdot \quad (3)$$

$$\delta_2 = (1 - x)\theta H \quad \cdot \quad (4)$$

[0035]

The plastic rotation angles θ_1 and θ_2 (radian) generated in the yield hinge line of the flange 13 of the steel column 12 can be expressed by the following Expressions (5) and (6).

[0036]

[Math 3]

$$\theta_1 = \frac{\delta_1}{y} = \frac{x\theta H}{y} \quad \cdot \quad (5)$$

$$\theta_2 = \frac{\delta_2}{z} = \frac{(1 - x)\theta H}{z} \quad \cdot \quad (6)$$

[0037]

[2. Bending Moment at Collapse Mechanism]

FIG. 8 shows actual stress-strain characteristics of a steel, such as the steel column 12 and the beam 21. In FIG. 8, the horizontal axis indicates strain of a steel, and the vertical axis indicates stress acting on the steel. The steel has an elastic region R1 in which stress increases in proportion to an increase of strain from a state where the strain is zero. A range having strain greater than that of the elastic region R1 is a non-elastic region R2. In the non-elastic region R2, the increasing rate of stress is lower than that in the elastic region R1. Stress at the boundary between the elastic region R1 and the non-elastic region R2 is yield stress σ_1 .

In the non-elastic region R2, stress has the maximum value at maximum stress

σ_2 . If strain becomes greater than the strain corresponding to the maximum stress σ_2 , stress becomes lower than the maximum stress σ_2 . A steel breaks at strain ε_1 .

[0038]

In contrast, in the present embodiment, when a theoretical solution is obtained by using a technique of limit analysis, a model having a rigid plasticity relationship as indicated with a line L1 in FIG. 9 is assumed as the stress-strain characteristics of the steel column 12 and the beam 21. In FIG. 9, the horizontal axis indicates strain of a steel, and the vertical axis indicates stress acting on the steel.

In this model, stress increases while strain remains zero. When the stress becomes the yield stress σ_1 , the steel yields. After the steel has yielded, strain increases without having a change in stress. In this model, strain hardening is not considered.

[0039]

Next, bending moment at collapse mechanism will be described in detail.

Yield bending moment M^0 (N) per unit length of a yield hinge line of the flange 13 of the steel column 12, and a yield axial force N^0_c (N/mm) per unit length of a discontinuous line generated in the web 14 of the steel column 12 are respectively given by the following Expressions (7) and (8).

Here, the plate thickness of the flange 13 of the steel column 12 is referred to as t_{cf} (mm), the plate thickness of the web 14 of the steel column 12 is referred to as t_{cw} (mm), the yield strength of the flange 13 of the steel column 12 is referred to as σ_{cfy} (N/mm²), and the yield strength of the web 14 of the steel column 12 is referred to as σ_{cwy} (N/mm²).

[0040]

[Math 4]

$$M^0 = \frac{\sigma_{cy} t_{cw}^2}{4} \quad \cdot \quad (7)$$

$$N_c^0 = \sigma_{cy} t_{cw} \quad \cdot \quad (8)$$

[0041]

Internal work W_{cf} due to the out-of-plane deformation of the flange 13 of the steel column 12 is given by the following Expression (9), as the sum of work of plastic rotation of each yield hinge line. In addition, internal work W_{cw} due to the local yield of the web 14 of the steel column 12 is given by the following Expression (10), as the sum of work of a plastic flow generated on each of the discontinuous lines.

Internal work W_{RC1} due to the cone failure generated in the cover concrete 16a around the flange 23₁ on the tension side of the beam 21 is given by the following Expression (11). Internal work W_{RC2} due to the bearing failure of the concrete 16 on the inner side around the flange 23₂ on the compression side of the beam 21 is given by the following Expression (12).

Here, the width of the flange 13 of the steel column 12 is referred to as B_c (mm), the cover thickness of the concrete 16 with respect to the flange 13 of the steel column 12 is referred to as d (mm) (refer to FIG. 2), the compression strength (design standard strength) of the concrete 16 is referred to as F_c (N/mm²), and the bearing coefficient of the concrete 16 is referred to as $\lambda(-)$ (in the present embodiment, 1.5). Compression strength F_c of the concrete 16 can be obtained by the “compression strength test method for concrete” disclosed in JIS A1108.

[0042]

[Math 5]

$$W_{cf} = 2M^0 B_c \left\{ 1 + \frac{xH}{y} + \frac{(1-x)H}{z} \right\} \theta \quad \cdot \quad (9)$$

$$W_{cw} = \frac{1}{2} N^0_c H [x(y + xH) + (1-x)\{z + (1-x)H\}] \theta \quad \cdot \quad (10)$$

$$W_{RC1} = xH \{ 2d(B_c + y + xH) + ml^2 \} \cdot 0.31 \sqrt{F_c} \theta \quad \cdot \quad (11)$$

$$W_{RC2} = \frac{B_c \{ z + (1-x)H \}}{2} \lambda F_c (1-x) H \theta \quad \cdot \quad (12)$$

[0043]

According to the principle of virtual work, bending moment at collapse mechanism M (N mm) with respect to the joint portion 31 is given by the following Expression (13). That is, a first equation based on the following Expression (13), in which the sum of the internal work W_{cf} , the internal work W_{cw} , the internal work W_{RC1} , and the internal work W_{RC2} is equal to the product of the bending moment at collapse mechanism M of the joint portion 31 and the rotation angle θ of the joint portion 31 with respect to the bending moment of the end portion of the beam 21, is derived.

Full plastic bending moment jM_p (N mm), which is the minimum value of the bending moment at collapse mechanism M, is obtained by simultaneously solving the following Expression (14) and can be given by the following Expressions (15) to (18).

The following Expression (14) is an equation expressing that the values obtained by partially differentiating the bending moment at collapse mechanism M by the variables x, y, and z are equal to zero. The variables x, y, and z are obtained by the following Expressions (16) to (18).

The full plastic bending moment jM_p indicates the bending moment when the steel has yielded in FIG. 9 and corresponds to the full plastic strength of the joint portion 31. Basically, the variable y is greater than the variable z. The reason is that as the bearing strength of concrete becomes greater than the cone failure strength as described above. In a case where the compression strength F_c of the concrete 16 is

zero (in a case of a column having a steel framed structure in which the column 11 does not include the reinforcing rods 15 and the concrete 16), the variable y and the variable z become equal to each other.

[0044]

[Math 6]

$$M\theta = W_{cf} + W_{cw} + W_{RC1} + W_{RC2} \quad \cdot \quad (13)$$

$$\frac{\partial M}{\partial x} = 0, \quad \frac{\partial M}{\partial y} = 0, \quad \frac{\partial M}{\partial z} = 0 \quad \cdot \quad (14)$$

$$M_p = 2M^0 B_c \left\{ 1 + \frac{xH}{y} + \frac{(1-x)H}{z} \right\} + \frac{1}{2} N_c^0 H \{ x(y+xH) + (1-x)\{z+(1-x)H\} \} \\ + xH \{ 2d(B_c + y + xH) + \pi d^2 \} \cdot 0.31 \sqrt{F_c} + \frac{B_c \{ z + (1-x)H \}}{2} \lambda F_c (1-x)H \quad \cdot \quad (15)$$

$$x = \frac{\frac{N_c^0 H}{2} (-y+z+2H) - 0.31 \sqrt{F_c} dH (2y+2B_c + \pi d) + \frac{B_c H \lambda F_c}{2} (z+2H) - 2M^0 B_c H \left(\frac{1}{y} - \frac{1}{z} \right)}{2N_c^0 H^2 + 4 \cdot 0.31 \sqrt{F_c} dH^2 + B_c H^2 \lambda F_c} \quad \cdot \quad (16)$$

$$y = \sqrt{\frac{2M^0 B_c}{2d \cdot 0.31 \sqrt{F_c} + \frac{N_c^0}{2}}} \quad \cdot \quad (17)$$

$$z = \sqrt{\frac{2M^0 B_c}{\frac{B_c \lambda F_c}{2} + \frac{N_c^0}{2}}} \quad \cdot \quad (18)$$

[0045]

When the above Expression (17) is arranged by substituting the yield bending moment M^0 given by the above Expressions (7) and (8), and the yield axial force N_c^0 per unit length, the following Expression (19) is obtained.

In a case of a column having a steel framed structure in which the column 11 does not include the reinforcing rods 15 and the concrete 16, the following Expression (19) may be calculated while having the compression strength F_c of the concrete 16 and the cover thickness d of the concrete 16 as zero.

[0046]

[Math 7]

$$y = B_c \sqrt{\frac{\sigma_{cy}}{\sigma_{cwy}}} \cdot \sqrt{\frac{t_g/t_{cw}}{2(B_c/2t_g)}} \cdot \sqrt{\frac{1}{1+1.24d\sqrt{F_c/(\sigma_{cwy}t_{cw})}}} \cdot (19)$$

[0047]

As described above, only the flange 13 of the steel column 12 corresponding to each of the range of the variable y above the beam 21 indicated by the plastic hinge, the range of the depth of the beam 21, and the range of the variable z below the beam 21 is pulled or compressed. Basically, the variable y is greater than the variable z. The range of a value obtained by the expression of (H_b+2y) having a center P1 as the center of the range is greater than the range of a value obtained by the expression (H_b/2+y) above the center P1 of the depth of the beam 21, and the range of a value obtained by the expression (H_b/2+z) below the center P1. Therefore, it is ascertained that even if a load acts on the beam 21 from above and below, only the flange 13 of the steel column 12 in the range of a value obtained by the expression (H_b+2y) having the center P1 of the depth of the beam 21 (refer to FIG. 10) as the center of the range is pulled by the beam 21.

[0048]

Therefore, as shown in FIG. 10, in the member length direction of the steel column 12, in a range R6 including a range in which the flange 13 of the steel column 12 is pulled by the beam 21, the flange 13 and the web 14 of the steel column 12 are joined to each other by the full strength joint portion 17. On the other hand, in the member length direction of the steel column 12, in a range R7 other than the range R6, that is, in a range R7 in which the flange 13 and the web 14 of the steel column 12 are not joined to each other by the full strength joint portion 17, the flange 13 and the web 14 of the steel column 12 are joined to each other by the non-full strength joint portion

18. In FIG. 10, the reinforcing rods 15 of the column 11 and the concrete 16 are shown in a manner of being seen through.

Here, the range R6, a range of a value obtained by the expression (H_b+2W) having the center P1 of the depth of the beam 21 as the center of the range. Then, W is equal to or greater than y obtained by the above Expression (19).

[0049]

In a design method for the beam-to-column connection 1 of the present embodiment, in the steel column 12, the range in which the web 14 and the flange 13 to which the beam 21 is joined are subjected to the full strength joining is set to the range of a value obtained by the expression (H_b+2W) having the center P1 of the depth of the beam 21 as the center of the range. In addition, in the steel column 12, a range other than the range described above (range in which the web 14 and the flange 13 to which the beam 21 is joined are not subjected to the full strength joining) is set for non-full strength joining (joined by a method other than full strength joining).

That is, in the beam-to-column connection 1, in a case of being seen along the member length direction, the full strength joint portion 17 is formed in at least the range of a value obtained by the expression (H_b+2y) having the center P1 of the depth of the beam 21 (refer to FIG. 10) as the center of the range, and the non-full strength joint portion 18 is formed in a range other than the range described above. From a viewpoint of further reducing the manufacturing cost, it is preferable that the full strength joint portion 17 is formed in the range of a value obtained by the expression (H_b+2y) having the center P1 of the depth of the beam 21 (refer to FIG. 10) as the center of the range, and the non-full strength joint portion 18 is formed in a range other than the range described above. In this case, the range in which the web 14 and the flange 13 are joined to each other by full strength joining can be only a range in which the

flange 13 is pulled due to a load acting on the beam from above and below. Therefore, breaking of the joint portion between the web 14 and the flange 13 is restrained, and an increase in manufacturing cost can be further restrained.

[0050]

As described above, the variable W is a value equal to or greater than the variable y obtained by the above Expression (19). The variable W may be longer than the variable y by 50 (mm) or longer, for example. In addition, a value obtained by the expression (H_b+2W) may range from 1.1 times to 1.2 times a value obtained by the expression (H_b+2y) , for example.

In addition, at the time of starting arc welding, sometimes outside air enters the inside of a shield used for arc welding. Therefore, sparks of arc welding are unlikely to be stable and proof stress of a part in which the arc welding has started is not stable. Thus, it is preferable to provide an approach section when starting arc welding.

In addition, it is preferable that the end portion of the web 14 or the like is subjected to groove cutting in advance in the range $R6$ in which the full strength joint portion 17 is formed. Moreover, it is preferable that an ultrasonic flaw detecting test or the like is performed with respect to this range $R6$ to check that an incomplete penetration of the web 14 and the flange 13 is not formed.

[0051]

As described above, according to the beam-to-column connection 1 of the present embodiment and the design method for the beam-to-column connection 1, even if a load acts on the beam 21 from above and below, only the flange 13 of the steel column 12 within the range of a value obtained by the expression (H_b+2y) having the center of the depth of the beam 21 as the center of the range is pulled by the beam 21. Therefore, in the member length direction of the steel column 12, the flange 13 and the

web 14 in at least the range, in which the flange 13 of the steel column 12 is pulled, are subjected to the full strength joining, so that the incomplete penetration of the web 14 and the flange 13 is not formed in the steel column 12 in this range. Then, the web 14 and the flange 13 of the steel column 12 in the remaining range are subjected to non-full strength joining.

Accordingly, when a load acts on the beam 21 and bending moment is generated, breaking of a weld joining the web 14 and the flange 13 to each other can be restrained in the steel column 12. In addition, compared to the case where the web 14 and the flange 13 are entirely subjected to the full strength joining, the manufacturing cost can be reduced.

Accordingly, breaking of a weld joining the web 14 and the flange 13 to each other is restrained, and an increase in manufacturing cost of the beam-to-column connection 1 can be suppressed by performing full strength joining of the web 14 and the flange 13 of the steel column 12 in a necessary range and narrowing the range of full strength joining.

[0052]

The flange 13 and the web 14 may be subjected to the full strength joining in a range in which tensile stress acts on the flange 13 of the steel column 12, and the flange 13 and the web 14 may be joined to each other by a method other than full strength joining in the range in which the flange 13 and the web 14 are not subjected to the full strength joining. The range in which tensile stress acts on the flange 13 of the steel column 12 and local yield is generated in the web 14 of the steel column 12 becomes a range R9 as shown in FIG. 5. That is, the range is a range from the plastic hinge 13₁ to the neutral axis C1 in the member length direction of the steel column 12.

Since the flange 13 and the web 14 of the steel column 12 are subjected to the

full strength joining within the range in which tensile stress is acting on the flange 13 of the steel column 12, the incomplete penetration of the web 14 and the flange 13 is not formed in the steel column 12 within this range R9. Within the range R9 in which tensile stress is acting on the flange 13, a weld is likely to break while having the incomplete penetration as an origin. However, since the incomplete penetration is not formed within this range R9, a weld between the web 14 and the flange 13 in the steel column 12 is unlikely to break when a load acts on the beam 21.

Accordingly, the manufacturing cost of the beam-to-column connection 1 can be suppressed by performing full strength joining in a necessary range and narrowing the range of full strength joining.

[0053]

(Second Embodiment)

Next, a second embodiment of the present invention will be described with reference to FIG. 11. The same reference signs are applied to the same constituent elements as the constituent elements described above, and a duplicated description thereof will be omitted below.

In the present embodiment, a theoretical solution is obtained by using the technique of limit analysis as in the first embodiment limit analysis, and the parameters such as the thickness of the beam-to-column connection 1 are limited to those which are frequently used, such that the range for full strength joining can be easily obtained.

[0054]

Specifically, there are provided limitations (i) to (iii), and assumption (iv) as follows.

(i) The yield strength σ_{cfy} of the flange 13 of the steel column 12 and the yield strength σ_{cwy} of the web 14 of the steel column 12 are equal to each other.

(ii) The ratio of a width B_c of the flange 13 of the steel column 12 to twice the plate thickness t_{cf} of the flange 13 of the steel column 12 is 3.0 or greater.

(iii) The ratio of the plate thickness t_{cf} of the flange 13 of the steel column 12 to the plate thickness t_{cw} of the web 14 of the steel column 12 is 3.33 or smaller.

(iv) The influence of cover concrete is disregarded (F_c is set to zero).

[0055]

It is ascertained that the influence of cover concrete with respect to the frequently used parameters of the beam-to-column connection 1 is approximately 10%.

The following Expression (20) is obtained by changing the above Expression (19) using the limitation (i) and the assumption (iv).

That is, it is ascertained that the variable y is expressed by the width B_c of the flange 13 of the steel column 12, the ratio of the plate thickness t_{cf} of the flange 13 of the steel column 12 to the plate thickness t_{cw} of the web 14 of the steel column 12, and the ratio of the width B_c of the flange 13 of the steel column 12 to twice the plate thickness t_{cf} of the flange 13 of the steel column 12.

[0056]

[Math 8]

$$y = B_c \cdot \sqrt{\frac{t_{cf}}{t_{cw}} \cdot \frac{B_c}{2t_{cf}}} \quad (20)$$

[0057]

For the ratio of the plate thickness t_{cf} of the flange 13 of the steel column 12 to the plate thickness t_{cw} of the web 14 of the steel column 12, the upper limit value 3.33 in the limitation (iii) is substituted in the above Expression (20). For the ratio of the

width B_c of the flange 13 of the steel column 12 to twice the plate thickness t_{cf} of the flange 13 of the steel column 12, the lower limit value 3.0 in the limitation (ii) is substituted in the above Expression (20). Accordingly, the following Expression (21) is obtained.

In the following Expression (20), if the lower limit value 3.0 of a value obtained by the expression $(B_c/(2t_{cf}))$ as the limitation (ii), and the upper limit value 3.33 of a value obtained by the expression (t_{cf}/t_{cw}) as the limitation (iii) are substituted in the following Expression (20), approximately $0.75 B_c$ is obtained as the maximum value of the variable y in the limitations (ii) and (iii).

[0058]

[Math 9]

$$y = B_c \cdot \sqrt{\frac{3.33}{2 \cdot 3.0}} = 0.75 B_c \quad \cdot (21)$$

[0059]

That is, in the present embodiment, the range R6 of the full strength joint portion 17 described above is set to the range of a value obtained by the expression (H_b+2W) having the center P1 of the depth of the beam 21 as the center of the range. However, W is equal to or greater than y obtained by the above Expression (21).

[0060]

In a case where the ratio of the width B_c of the flange 13 of the steel column 12 to twice the plate thickness t_{cf} of the flange 13 of the steel column 12 is set to 2.0 or greater, and the ratio of the plate thickness t_{cf} of the flange 13 of the steel column 12 to the plate thickness t_{cw} of the web 14 of the steel column 12 is set to 4.0 or smaller, the value of the variable y becomes $1.0 B_c$. In a case where the ratio of the width B_c of the flange 13 of the steel column 12 to twice the plate thickness t_{cf} of the flange 13 of the

steel column 12 is set to 5.0 or greater, and the ratio of the plate thickness t_{cf} of the flange 13 of the steel column 12 to the plate thickness t_{cw} of the web 14 of the steel column 12 is set to 2.5 or smaller, the value of the variable y becomes $0.5 B_c$.

[0061]

As described above, according to the beam-to-column connection 1 of the present embodiment, for the frequently used parameters such as the plate thickness, the range in which the web 14 and the flange 13 of the steel column 12 are subjected to the full strength joining is determined by only the width B_c of the flange 13 of the steel column 12 and the depth H_b of the beam 21. Therefore, the range in which the web 14 and the flange 13 of the steel column 12 are subjected to the full strength joining can be easily obtained.

FIG. 11 verifying the accuracy of evaluation in which the variable y is set to $0.75 B_c$ in the present embodiment will be described in a third embodiment.

[0062]

(Third Embodiment)

Next, the third embodiment of the present invention will be described with reference to FIGS. 12 to 20. The same reference signs are applied to the same constituent elements as the constituent elements described above, and a duplicated description thereof will be omitted below.

In the first embodiment and the second embodiment, a theoretical solution is obtained by using the technique of limit analysis, and the range of the full strength joint portion 17 is set based on a plasticizing range of the web 14 of the column 11 of the full plastic bending moment jM_p in the joint portion 31 between the column 11 and the beam 21. However, there are cases where the plasticizing range is widened due to strain hardening of a steel after the joint portion 31 has yielded.

In contrast, in the present embodiment, conditions to have a sufficient range of the full strength joint portion 17 as described above even in a case where the joint portion 31 is significantly deformed and the plasticizing range of the joint portion 31 is widened will be examined. In the present embodiment, a distribution range of plastic strain corresponding to the variable y is obtained by finite element analysis.

[0063]

FIGS. 12 to 14 show analytic models used in the finite element analysis. The analytic model in FIG. 12 is a 1/4 model expressing the beam-to-column connection 1. In this 1/4 model, a first surface S1 and a second surface S2 are set to be symmetric surfaces of the analysis condition.

The analytic model in FIG. 12 shows a case where the column 11 of the beam-to-column connection 1 has a steel framed structure and the steel column 12 has an H-shaped cross section.

[0064]

The conditions of the finite element analysis are as follows.

(I) The element of the analytic model is set to be an eight-nodal point solid element.

(II) A reinforcement height of a weld metal portion of a part in which the end portion of the beam 21 is welded to the steel column 12 is set to be 1/4 of the plate thickness of the flange 23 of the beam 21. However, in a case where the plate thickness of the flange 23 of the beam 21 is 40 mm or greater, the reinforcement height is set to 10 mm.

(III) In the weld metal portion between the web 14 and the flange 13 of the steel column 12, both a leg length and the reinforcement height are set to 10 mm.

[0065]

(IV) As the material characteristics of the steel column 12 and the beam 21, stress-strain characteristics in which a tensile test result of SN490 (rolling steel for building construction) is modeled are used. FIG. 15 shows stress-strain characteristics used in the elements of the steel column 12 and the beam 21. In FIG. 15, the horizontal axis indicates strain, and the vertical axis indicates stress. In FIG. 15, a one-dot chain line L6 is a line indicating average stress-average strain of the tensile test result. In addition, a solid line L7 is a line in which the average stress-average strain of the tensile test result is modeled. In addition, a dotted line L8 is a line in which the modeled average stress-average strain is converted into true stress-true strain. The characteristics of the average stress-average strain indicated by the dotted line L8 are used in the finite element analysis.

[0066]

(V) The stress-strain characteristics in which the tensile test result of a YGW18 wire is modeled are used as the material characteristics of each weld metal portion. FIG. 16 shows the stress-strain characteristics used in the element of the weld metal portion. In FIG. 16, the horizontal axis indicates strain, and the vertical axis indicates stress. In FIG. 16, a one-dot chain line L11 is a line indicating the average stress-average strain of the tensile test result. In addition, a solid line L12 is a line in which the average stress-average strain of the tensile test result is modeled. In addition, a dotted line L13 is a line in which the modeled average stress-average strain is converted into the true stress-true strain. The characteristics of the average stress-average strain indicated by the dotted line L13 are used in the finite element analysis.

(VI) The yield strength of the steel column 12 and the beam 21 is set to 380 MPa, and the tensile strength is set to 519 MPa.

(VII) The yield strength of the weld metal portion is set to 526 MPa, and the

tensile strength is set to 606 MPa.

(VIII) A load F (refer to FIG. 12) is caused to act on an end of the beam 21 opposite to the steel column 12, such that a load condition of applying unidirectionally forced displacement in the vertical direction is realized.

[0067]

Table 1 shows the list of the ratios of the analysis variables of the analytic models. The parameters of the dimensions of the steel column 12 and the beam 21 are set to be combinations of the values shown in Table 1 of the frequently used ratios.

That is, the values of the ratios are determined as follows.

(a) The ratio of the width B_c to the plate thickness t_{cf} in the flange 13 of the steel column 12 ranges from 3.0 to 8.0.

(b) The ratio of the width to the plate thickness t_{cw} of the web 14 of the steel column 12 (width-thickness ratio) is 12.0 or greater, in a case where the steel column 12 has a flanged cruciform cross section. In a case where the steel column 12 has an H-shaped cross section, this ratio is 24.0 or greater.

(c) The ratio of a depth H_c to the width B_c of the flange 13 in the steel column 12 ranges from 1.7 to 3.5.

(d) The ratio of the depth H_b of the beam 21 to the depth H_c of the steel column 12 ranges from 0.7 to 1.5.

(e) The ratio of a width B_b of the flange 23 of the beam 21 to the width B_c of the flange 13 of the steel column 12 ranges from 0.5 to 1.0.

(f) The ratio of a plate thickness t_{bw} of the web 24 of the beam 21 to the plate thickness t_{cw} of the web 14 of the steel column 12 ranges from 0.5 to 1.0.

[0068]

[Table 1]

Ratio of width to plate thickness of steel column		Ratio of depth to width of flange in steel column	Ratio of plate thickness of web of beam to plate thickness of web of steel column	Ratio of width of flange of beam to width of flange of steel column	Ratio of depth of beam to depth of steel column
Flange	Web				
		H_c/B_c	t_{bw}/t_{cw}	B_b/B_c	H_b/H_c
3.0	12.0	1.7	0.5	0.5	0.7
8.0	32.0	3.5	1.0	1.0	1.5

[0069]

Specifically, the dimensions are determined as follows.

(g) The depth H_c of the steel column 12 is set to a fixed value of 600 mm.

(h) The width B_c of the flange 13 in the steel column 12 is determined depending on the variables of the ratio of the depth H_c to the width B_c of the flange 13 in the steel column 12.

(i) The plate thickness t_{cw} of the web 14 and the plate thickness t_{cf} of the flange 13 are determined depending on the variables of the ratio of the width to the plate thickness t_{cw} in the web 14 of the steel column 12, and the ratio of the width B_c to the plate thickness t_{cf} in the flange 13 of the steel column 12.

(j) The depth H_b of the beam 21 and the width B_b of the flange 23 of the beam 21 are determined depending on the variables of the ratio of the depth H_b of the beam 21 to the depth H_c of the steel column 12, and the ratio of the plate thickness t_{bw} of the web 24 of the beam 21 to the plate thickness t_{cw} of the web 14 of the steel column 12.

[0070]

(k) The plate thickness t_{bw} of the web 24 of the beam 21 is determined depending on the variables of the ratio of the width B_b of the flange 23 of the beam 21 to the width B_c of the flange 13 of the steel column 12.

(l) The plate thickness t_{bf} of the flange 23 of the beam 21 is determined such

that bending proof stress of the beam 21 with respect to local proof stress of the joint portion 31 becomes 1.2 times or greater. In this case, in a case where the ratio of the width B_b to the plate thickness t_{bf} of the flange 23 of the beam 21 is in an FD rank, the plate thickness t_{bf} of the flange 23 of the beam 21 is increased to have the value of the boundary between the FD rank and an FC rank. In addition, in a case where the plate thickness t_{bf} of the flange 23 of the beam 21 becomes smaller than the plate thickness t_{bw} of the web 24 of the beam 21, the plate thickness t_{bf} of the flange 23 of the beam 21 is set to be equal to the plate thickness t_{bw} of the web 24 of the beam 21.

(m) In a case where the ratio of the plate thickness t_{cf} of the flange 13 to the plate thickness t_{cw} of the web 14 in the steel column 12 exceeds 3.33, the analysis case is excluded. The excluded analysis case is a case in which the frequency in use is low.

[0071]

Tables 2 and 3 show Analysis Case 1 to Analysis Case 56 in the list of the parameters of the dimensions of the steel column 12 and the beam 21 determined by the procedures described above.

In Tables 2 and 3, the analysis case number in which the ratio of the width to the plate thickness t_{cw} in the web 14 of the steel column 12 is 12.0 indicates a case where the steel column 12 has a flanged cruciform cross section. In addition, the analysis case number in which the ratio of the width to the plate thickness t_{cw} in the web 14 of the steel column 12 is 32.0 indicates a case where the steel column 12 has an H-shaped cross section.

[0072]

[Table 2]

Analysis case number	Steel column				Beam				Analysis variables					Ratio of width of flange of beam to width of flange of steel column	Ratio of depth of beam to depth of steel column						
	Depth	Width of flange	Plate thickness of web	Plate thickness of flange	Depth	Width of flange	Plate thickness of web	Plate thickness of flange	Ratio of depth to width of flange in steel column	Ratio of plate thickness of web of beam to plate thickness of web of steel column	Ratio of width of flange of beam to width of flange of steel column										
												H _c	B _c			t _{ew}	t _{ef}	H _b	B _b	t _{bw}	t _{bf}
	mm	mm	mm	mm	mm	mm	mm	mm	-	-	-	-	-	-							
1	600	353	20.1	58.8	900	353	20.1	20.1	3.0	12.0	1.7	1.0	1.0	1.5							
2	600	353	23.2	22.1	900	353	23.2	23.2	8.0	12.0											
3	600	353	8.7	22.1	900	353	8.7	13.4	8.0	32.0											
4	600	171	22.6	28.6	900	171	22.6	22.6	3.0	12.0	3.5	1.0	1.0	1.5							
5	600	171	24.1	10.7	900	171	24.1	24.1	8.0	12.0											
6	600	171	8.5	28.6	900	171	8.5	8.5	3.0	32.0											
7	600	171	9.0	10.7	900	171	9.0	9.0	8.0	32.0	1.7	0.5	1.0	1.5							
8	600	353	20.1	58.8	900	353	10.0	22.4	3.0	12.0											
9	600	353	23.2	22.1	900	353	11.6	13.4	8.0	12.0											
10	600	353	8.7	22.1	900	353	4.3	13.4	8.0	32.0	3.5	0.5	1.0	1.5							
11	600	171	22.6	28.6	900	171	11.3	26.5	3.0	12.0											
12	600	171	24.1	10.7	900	171	12.1	20.3	8.0	12.0											
13	600	171	8.5	28.6	900	171	4.2	12.1	3.0	32.0	1.7	1.0	1.0	1.5							
14	600	171	9.0	10.7	900	171	4.5	8.2	8.0	32.0											
15	600	353	20.1	58.8	900	176	20.1	34.8	3.0	12.0											
16	600	353	23.2	22.1	900	176	23.2	23.2	8.0	12.0	1.7	1.0	1.0	1.5							
17	600	353	8.7	22.1	900	176	8.7	8.7	8.0	32.0											
18	600	171	22.6	28.6	900	86	22.6	25.8	3.0	12.0											
19	600	171	24.1	10.7	900	86	24.1	24.1	8.0	12.0	3.5	1.0	1.0	1.5							
20	600	171	8.5	28.6	900	86	8.5	13.7	3.0	32.0											
21	600	171	9.0	10.7	900	86	9.0	9.0	8.0	32.0											
22	600	353	20.1	58.8	900	176	10.0	47.5	3.0	12.0	1.7	0.5	1.0	1.5							
23	600	353	23.2	22.1	900	176	11.6	27.5	8.0	12.0											
24	600	353	8.7	22.1	900	176	4.3	12.7	8.0	32.0											
25	600	171	22.6	28.6	900	86	11.3	59.3	3.0	12.0	3.5	0.5	1.0	1.5							
26	600	171	24.1	10.7	900	86	12.1	45.3	8.0	12.0											
27	600	171	8.5	28.6	900	86	4.2	25.2	3.0	32.0											
28	600	171	9.0	10.7	900	86	4.5	16.9	8.0	32.0	1.7	0.5	1.0	1.5							

[Table 3]

Analysis case number	Steel column				Beam			Analysis variables				
	Depth	Width of flange	Plate thickness of web	Plate thickness of flange	Depth	Width of flange	Plate thickness of web	Plate thickness of flange	Ratio of width to plate thickness of steel column		Ratio of depth of beam to depth of steel column	Ratio of width of flange of beam to width of flange of steel column
	H _c mm	B _c mm	t _{ew} mm	t _{ef} mm	H _b mm	B _b mm	t _{bw} mm	t _{bf} mm	Flange	Web		
29	600	353	20.1	58.8	420	353	20.1	20.1	-	-	-	-
30	600	353	23.2	22.1	420	353	23.2	23.2	8.0	12.0	1.7	1.0
31	600	353	8.7	22.1	420	353	8.7	13.4	8.0	32.0		
32	600	171	22.6	28.6	420	171	22.6	22.6	3.0	12.0	3.5	1.0
33	600	171	24.1	10.7	420	171	24.1	24.1	8.0	12.0		
34	600	171	8.5	28.6	420	171	8.5	8.5	3.0	32.0	8.0	1.0
35	600	171	9.0	10.7	420	171	9.0	9.0	8.0	32.0		
36	600	353	20.1	58.8	420	353	10.0	21.9	3.0	12.0	1.7	0.5
37	600	353	23.2	22.1	420	353	11.6	13.4	8.0	12.0		
38	600	353	8.7	22.1	420	353	4.3	13.4	8.0	32.0	3.5	0.5
39	600	171	22.6	28.6	420	171	11.3	19.4	3.0	12.0		
40	600	171	24.1	10.7	420	171	12.1	12.1	8.0	12.0	1.7	0.5
41	600	171	8.5	28.6	420	171	4.2	9.9	3.0	32.0		
42	600	171	9.0	10.7	420	171	4.5	6.5	8.0	32.0	1.7	0.5
43	600	353	20.1	58.8	420	176	20.1	43.3	3.0	12.0		
44	600	353	23.2	22.1	420	176	23.2	23.2	8.0	12.0	3.5	1.0
45	600	353	8.7	22.1	420	176	8.7	8.7	8.0	32.0		
46	600	171	22.6	28.6	420	86	22.6	30.3	3.0	12.0	1.7	0.5
47	600	171	24.1	10.7	420	86	24.1	24.1	8.0	12.0		
48	600	171	8.5	28.6	420	86	8.5	15.7	3.0	32.0	3.0	0.5
49	600	171	9.0	10.7	420	86	9.0	9.0	8.0	32.0		
50	600	353	20.1	58.8	420	176	10.0	48.3	3.0	12.0	1.7	0.5
51	600	353	23.2	22.1	420	176	11.6	20.6	8.0	12.0		
52	600	353	8.7	22.1	420	176	4.3	10.7	8.0	32.0	3.5	0.5
53	600	171	22.6	28.6	420	86	11.3	44.5	3.0	12.0		
54	600	171	24.1	10.7	420	86	12.1	26.8	8.0	12.0	3.0	0.5
55	600	171	8.5	28.6	420	86	4.2	20.9	3.0	32.0		
56	600	171	9.0	10.7	420	86	4.5	10.9	8.0	32.0	-	-

[0073]

The finite element analysis is performed for 56 analysis cases in total as shown in Tables 2 and 3. In each of the analysis cases, the equivalent plastic strain of a weld toe portion 19a of a weld metal portion 19 shown in FIGS. 13 and 14 is obtained throughout the member length direction of the steel column 12.

FIG. 17 shows an example of a result of obtained plastic strain in the case of Analysis Case 2 (the depth H_b of the beam 21 is 900 mm). In FIG. 17, the horizontal axis indicates the equivalent plastic strain, and the vertical axis indicates the position with respect to the center of the depth of the beam 21. The upward direction is set as the positive side of the position.

In addition, in FIG. 17, a line L16 expressed by square plotting indicates a state when the yield of the joint portion 31 of the beam-to-column connection 1 has started. On the other hand, a line L17 expressed by rhombic plotting indicates a state when the end portion of the beam 21 is rotated by 0.02 radian (approximately 1.15°).

[0074]

Here, the time when the yield of the joint portion 31 has started is defined as a state when the inclination is reduced to 1/3 with respect to the inclination of the rotation angle-bending moment of the joint portion 31 in the initial stage. The state in which the end portion of the beam 21 is rotated by 0.02 (radian) denotes the maximum displacement generated in a building when an earthquake occurs.

[0075]

In both the line L16 and the line L17, it is ascertained that the equivalent plastic strain of the weld to portion 19a, that is, plastic strain of the web 14 of the steel column 12 is distributed about the position of the flange 23 of the beam 21.

It is ascertained that when the yield of the joint portion 31 starts, plastic strain

is distributed to a range X of approximately 105 mm outside the depth of the beam 21. On the other hand, it is ascertained that plastic strain is distributed to the range X of approximately 150 (mm) outside the depth of the beam 21 in the state where the end portion of the beam 21 is rotated by 0.02 (radian).

[0076]

FIG. 18 shows the result of the obtained distribution range X of plastic strain in each of the analysis cases in a state where the end portion of the beam 21 is rotated by 0.02 (radian). In FIG. 18, the horizontal axis indicates the width B_c of the flange 13 of the steel column 12, and the vertical axis indicates the distribution range X of plastic strain.

It is ascertained that the distribution range X of plastic strain is a range equal to or smaller than 1.1 times the width B_c in the flange 13 of the steel column 12.

Therefore, in the present embodiment, the range R6 of the full strength joint portion 17 described above is the range of a value obtained by the expression (H_b+2W) having the center P1 of the depth of the beam 21 as the center of the range. However, W is equal to or greater than 1.1 B_c . That is, in the analytic model having the beam-to-column connection 1, the weld metal portion 19 in the range of a value obtained by the expression (H_b+2W) having the center P1 of the depth of the beam 21 as the center of the range is adopted as the full strength joint portion 17, and the weld metal portion 19 other than the full strength joint portion 17 is adopted as the non-full strength joint portion 18.

[0077]

As described above, according to the beam-to-column connection 1 of the present embodiment, with the parameters such as the frequently used thickness while the steel column 12 has a flanged cruciform cross section or an H-shaped cross section,

plastic strain is distributed in the steel column 12 in a range equal to or smaller than the range of a value obtained by the expression $(H_b + 2.2 B_c)$ having the center of the depth of the beam 21 as the center of the range with respect to the member length direction of the steel column 12. When the flange 13 and the web 14 in the distribution range of plastic strain in the steel column 12 are subjected to the full strength joining, the incomplete penetration of the web 14 and the flange 13 is not formed in the steel column 12 within this range. The web 14 and the flange 13 of the steel column 12 in the remaining range are joined to each other by non-full strength joining. Therefore, a weld joining the web 14 and the flange 13 to each other is unlikely to break in the steel column 12 when a load acts of the beam 21.

In addition, the range in which the web 14 and the flange 13 are subjected to the full strength joining in the steel column 12 is determined by only the width B_c of the flange 13 of the steel column 12 and the depth H_b of the beam 21. Accordingly, the range in which the web 14 and the flange 13 of the steel column 12 are subjected to the full strength joining can be easily obtained.

[0078]

FIG. 19 shows the result of the obtained distribution range X of plastic strain in each of the analysis cases in a state where the end portion of the beam 21 is rotated by 0.03 (radian). In addition, FIG. 20 shows the result of the obtained distribution range X of plastic strain in each of the analysis cases in a state where the end portion of the beam 21 is rotated by 0.04 (radian).

It is ascertained that the distribution range X of plastic strain is the range equal to or smaller than 1.2 times the width B_c in the flange 13 of the steel column 12 in both the state of being rotated by 0.03 (radian) and the state of being rotated by 0.04 (radian).

That is, it is postulated that even if the rotation angle of the end portion of the

beam 21 is increased, the distribution range X of plastic strain is saturated in the range equal to or smaller than 1.2 times the width B_c . In this case, it is ascertained that the structure can sufficiently withstand a large earthquake by performing full strength joining in at least the range of a value obtained by the expression $(H_b + 2.4 B_c)$ having the center of the depth H_b of the beam 21 as the center of the range.

[0079]

Here, FIG. 11 of the second embodiment will be described. In the analysis cases shown in Tables 2 and 3, the distribution ranges X of plastic strain of 56 analysis cases corresponding to the second embodiment are obtained.

It is ascertained that the distribution range X of plastic strain is the range equal to or smaller than 0.75 times the width B_c in the flange 13 of the steel column 12. It is ascertained that the plasticizing range of the yield proof stress time in the result of the finite element analysis can be fully evaluated to be on the safe side.

[0080]

Hereinabove, each of the embodiments of the present invention has been described. However, these embodiments are presented as examples, and the scope of the present invention is not limited to only these embodiments. These embodiments can be realized in various other forms and can be subjected to various omissions, replacements, and changes within a range not departing from the gist of the invention. These embodiments and a modification thereof are included in the scope and the gist of the invention and are also included in the invention disclosed in Claims and the scope equivalent thereto in a similar manner.

[Brief Description of the Reference Symbols]

[0081]

1: beam-to-column connection

11: column

12: steel column

13, 23: flange

13₁, 13₂, 13₃, 13₄: plastic hinge

14, 24: web

17: full strength joint portion

18: non-full strength joint portion

21: beam

[Document Type] CLAIMS

What is claimed is:

1. A beam-to-column connection which is a structure of a non-diaphragm type, the beam-to-column connection comprising:
 - a column that has a steel reinforced concrete structure or a steel framed structure; and
 - a beam that has a steel framed structure joined to the column,wherein the column has a steel column which has a flanged cruciform cross section or an H-shaped cross section and includes a web, a flange, and a full strength joint portion and a non-full strength joint portion joining the web and the flange to each other, and
 - wherein the full strength joint portion is formed in at least a range in which tensile stress acts on the flange of the steel column when bending moment acts on the beam.
2. The beam-to-column connection according to Claim 1,
 - wherein the full strength joint portion is formed in a range of a value equal to or greater than H_b+2y obtained by using the following expression, while having a center of a depth of the beam as a center of the range,
 - wherein, in the following expression, B_c is a width (mm) of the flange of the steel column, t_{cf} is a plate thickness (mm) of the flange of the steel column, t_{cw} is a plate thickness (mm) of the web of the steel column, σ_{cfy} is yield strength (N/mm²) of the flange of the steel column, σ_{cwy} is yield strength (N/mm²) of the web of the steel column, H_b is a depth (mm) of the beam, F_c is compression strength (N/mm²) of concrete of the column, and d is a cover thickness (mm) of concrete with respect to the

flange of the steel column, and

wherein F_c and d are zero when the column has the steel framed structure.

[Math 1]

$$y = B_c \sqrt{\frac{\sigma_{cf}}{\sigma_{cwy}}} \cdot \sqrt{\frac{t_{cf}/t_{cw}}{2(B_c/2t_{cf})}} \cdot \sqrt{\frac{1}{1+1.24d\sqrt{F_c}/(\sigma_{cwy}t_{cw})}}$$