

Research on bearing behavior of the column composed of double limbs subjected to bending and compression in precipitator casing

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Abstract. Cross-section of the column in enclosure structure of electrostatic precipitator casing is composed of double H-shaped limbs because of the requirement of supporting the top beam with a wide box section and bearing large axial compression. For the failure modes and failure mechanism of the composite section column subjected to the combined loads of axial compression and bending, nonlinear finite element method was used to study the bearing behavior of the column in consideration of the initial imperfection. Besides, the influence of eccentricity ratio was investigated. The results indicate that there were three possible failure modes of the column. The first was the elasto-plastic flexural-torsional buckling occurring on the front flange closely below the top. The second was the elasto-plastic flexural-torsional buckling occurring on the front flange at the mid-span of the top column segment. The third was the strength failure occurring on the web at the first braced location where the stresses reached yielding because of significant not only compressive but also shear stresses. Buckling failure mainly occurred when eccentricity ratio was low and strength failure mainly occurred when eccentricity ratio was high.

Key words. Wall-column structure system of precipitator casing, Column composed of double limbs; Failure mechanism, Buckling; Nonlinear finite element method.

1. Introduction

Precipitator is the important environmental protection equipment which is widely used in electric power, metallurgy, building materials and other industries. Casing is the most crucial part of this process. Under the axial load and transversal uniform

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pressure, the H-shaped steel columns in exterior casing are flexural-compression members. Wallboards of the casing and column are connected by continuous welding to form an interactive structure system. Wallboard exerts stressed skin effect and bears load with column collaboratively, and provides transversal support to column to improve its stability^[1]. Much research has been investigated in theoretical and experimental method about stressed skin effect^[2-3]. NAGY Z^[4] investigated the influence of the diaphragm effect on the behavior of pitched roof portal frames, and the impact of stressed skin action on structural performance of pitched roof portal frames was obtained. Recently, the research on diaphragm-braced members is mainly concentrated on the light steel structure which used the skin plate as the profiled steel sheet. Electrostatic precipitator casing wallboard was a stiffened steel plate of greater stiffness. The column was a double limbs H-shaped steel column, and the two limbs were connected by connecting wallboards and channels. The angle stiffener on wallboards provides restraints to column by the diaphragm. The particularity of these structures makes that the previous research conclusions of the existing stressed skin structures is not fully applicable to the electrostatic precipitator casing wall-column structure system. Consequently, the failure mode and failure mechanism of the casing column composed of double limbs under the axial and transverse load were studied by nonlinear finite element method. Besides, most of the previous research only considered the imperfection on skeleton members but without the imperfection on diaphragm. However, this paper would take not only the imperfection on column but also that on wallboard into account.

2. Research model

2.1. Structural model

The geometrical configuration and parameters of the examined wall-column structural system in precipitator casing was based on practical projects. In practical electrostatic precipitator, one span wall plate corresponds to one electric field, and only two spans of wallboards were built in research model so as to improve modeling and computation efficiency. The geometrical structure of the structural system and the displacement coordinate system were shown in Figure 1. Column was the rolling H-shaped steel column. The middle column between the two cross-wallboards was the double limbs column, and the two limbs were connected by connecting wallboard with transversal angle steel stiffeners. For the H-shaped steel column, the x -axis in Figure 1 was strong stiffness axis, and the y -axis was weak stiffness axis. As a beam-column, the buckling of the column in the plane of the wallboard (around the y -axis, which moment acting outside the plane) would present the lateral flexural-torsional buckling. The stability of the middle double limbs column was studied in this paper. The wallboard and the side flange of the middle column were connected by continuous welding. The flange on the side of wallboard was defined as the rear flange and the column flange far from wallboard was the front flange. The web connecting the wallboard and the two limbs were connected by continuous welding. To reduce the calculated length, perpendicular to the wall plane, all the column were

arranged 4 middle braces at equal intervals along the height direction to provide the Y -support. The middle of the double limbs H-shaped steel column was set four connecting channels with large stiffness to provide X -support. The column spacing between the adjacent connecting channels was considered as a calculation span, which value was l_0 , and was the calculated length of the column around the y -axis.

2.2. Finite element model

In this paper, the finite element program ANSYS was used for nonlinear analysis. The material was an ideal elasto-plastic model. The casing was made of steel with the yielding strength $f_y=235\text{MPa}$. Elastic modulus $E=2.06\times 10^5\text{MPa}$. Poisson's ratio $\nu=0.3$. According to the actual constraints project in the bottom of the column, the middle was constrained in three directions translational degrees of freedom. The edge of the column only constrains the Z -translational degrees of freedom. To simulate the Y direction constraints provided by central braces on column, the Y direction of the translational constraints were applied on the corresponding height in the column position of the rear flange. In practical structure, the casing top wallboard was connected with the stiffened plate, and the bottom was connected with the stiffened hopper. Therefore, in finite element model, the top and bottom edge of the wallboard were applied the translation constrains that perpendicular to the wallboard(Y direction). All structure components were simulated by the Shell 181 element. Considering the geometric nonlinear effect, the structure response path was tracked by arc-length method.

2.3. Load

In practical projects, the casing column bore axial pressure which was transmitted by upper connecting beams on column top and transversal uniform loads combined with negative pressure and wind load, which were transmitted from wallboard casing. In calculation process of finite element method, a rigid cover plate was set on the top of each double limbs H-shaped steel column in the middle. A vertical uniform loading was applied on the rigid cover and the axial pressure N was transmitted to the column and adjacent wallboards through the cover plate. The wiring load q was applied on the central line of the front flange of the column. To induce the lateral flexural-torsional buckling of the column, while applying the transversal uniform load, a lateral disturbance load q_d was applied in the form of uniformly distributed load on the front flange side of the H-shaped steel column(as shown in Figure 1), q_d value was taken as $q/1000$.

In nonlinear calculation, the axial load and moment were loaded simultaneously. The moment of column section was defined as a positive number when the front flange in compression and as a negative number when the rear flange in compression. The load eccentricity was defined as $e=M_1/N$, $M_1=0.078ql_0^2$, which was the maximum positive moment of the five-span continuous beam under transversal uniform load independently. The survey of engineering design data of 89 electric precipitators with different regions and different installed capacity shows that the e

value range was 32.1-162.7. Therefore, when applying a flexural-compression load to the casing column, e was valued in the range of 30-180.

2.4. Validation of finite element method

In order to validate the accuracy of the finite element method in this paper, a finite element model was established according to the geometrical structure of the H1 specimen in reference^[10]. The steel was modeled by the ideal elasto-plastic model, and the nonlinear numerical simulation was carried out. The specimen was two Z-type purlins. The upper flange was connected with the profiled steel sheet. Purlins at both edges were simply supported. The specimen was loaded by stacking bricks on the profiled steel sheet. The maximum bending moment of the purlins was 14.66kN/m, while the finite element value was 13.10kN/m. The strain hardening of the material was not considered in the finite element calculation, and the practical structure had a certain plastic reserves. That may be the reason for the difference that calculated value was slightly lower than the experimental value. It can be considered that the finite element method of this paper can simulate the nonlinear response of wall-column structure system accurately.

2.5. Initial geometric imperfection

In order to induce the lateral flexural-torsional buckling of the column??considering the influence of the initial geometric imperfections, an initial geometric imperfections in form of a sinusoidal half-wave around the y -axis was applied in the connecting channel area of the middle two limbs of the H-shaped steel column(the connecting channel can be regarded as the lateral support of the H-shaped steel column). The defect form was shown in Figure 2. The defect amplitude δ was valued $H/1000$. H was the height of column.

In consideration of the influence of initial geometric imperfections of the wallboards and the connecting wallboards on column bearing capacity, the structural models of the sinusoidal half-wave defects in the support sections of the middle double limbs column was nonlinearly calculated in different eccentric e values of the axial load, with N and q applied simultaneously. When reaching the limit point, the larger deformation was the Y -direction deformation of the connecting wallboard near the column top area and wallboards adjacent to the column area. There was also a certain deflection(Y -direction) deformation and slight torsional deformation on column. When the load reaches the limit point, the deformation mode was taken as the limit point deformation defect mode of the structure system. The amplitude was valued $H/1000$. The defect mode consults of three defects: the integrated bending defect of column around y -axis and x -axis; the initial torsional defect introduced in column section; the defect of wallboard and connecting wallboard. Using this defect model to study the stability of the column was biased conservative. In the later analysis, for each of the structure the limit point deformation defect mode was firstly obtained based on different load eccentricity conditions. Then the defect model was calculated by nonlinear method to study the stability of the column at

different load eccentricity.

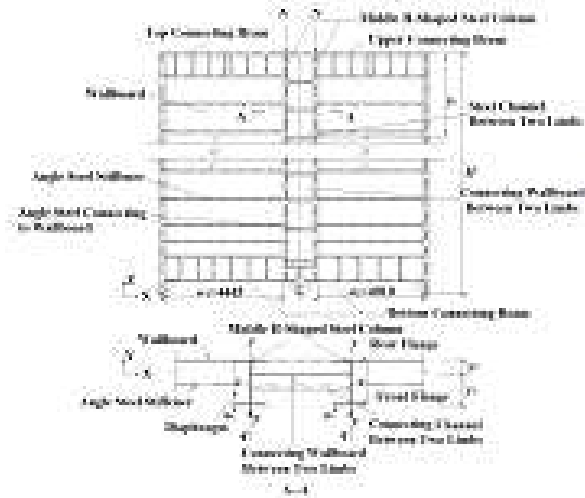


Fig. 1. Wall-column structure system of electrostatic precipitator casing and the coordinate system

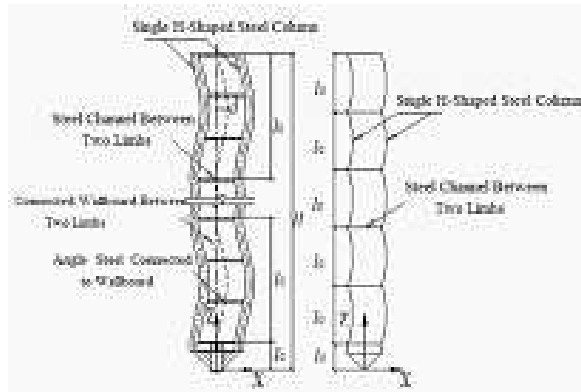


Fig. 2. Initial geometrical imperfection shape of the column

3. Failure modes of the column

The nonlinear finite element analyses were carried out for different configured column under different eccentricity conditions. The failure modes of the beam-column can be generalized into three kinds. The first one was in low e value, the structural response was similar to the axial compression column. For some of the column with smaller interval distance of braces, thicker wallboard and thinner flange, there would be slightly flexural-torsional buckling below the top of the column as

shown in Figure 3. Due to the small interval distance of braces, the bending moment produced by the transversal uniform load was small. And due to the thick wallboard, the bending ability of column which assisted by wallboard was strong. Therefore, the column deformation and stress caused by the bending moment were in a low level. The middle of the span with the maximum stress under the bending moment was not prone to buckling. The axial pressure of the H-shaped column under the column top was large, resulting in a large compressive stress. As the flange was weak, the front flange was prone to flexural-torsional buckling. When reaching the limit load, the deformed front flange has a considerable part reached the yield point. So this failure mode was named as elasto-plastic flexural-torsional buckling occurring on the front flange closely below the top of the front flange. Since there were several of restricted conditions, only few examples occurred such failure mode.

The second failure mode was the flexural-torsional buckling at the middle area of the first span of column front flange, as it shown in Figure 4. This was the most favorable failure mode in accordance with the common construction and constant load condition cases of electrostatic precipitator casing column. Under the interaction of axial force and bending moment, the maximum compressive stress was occurred on the front flange of the top of the mid-span, which was away from the horizontal connection of the constraints of the channel and prone to lateral flexural-torsional buckling. When reaching the limit loads there was a large part of the front side of the web and the front flange have reached yield point and buckling. Therefore, this failure mode was named as the elasto-plastic flexural-torsional buckling occurring on the front flange at the mid-span of the top column segment.

The third failure mode usually occurs when the e value was in a high level. The structure response was similar to the column under transversal uniform loadings. For the column with longer interval distance of braces, thinner wallboard, thicker flange and relatively weak web, the deformation mode was similar to the second failure mode. The lateral flexural-torsional deformation with small deformation amplitude would occur on the front flange portion of the mid-span region. The failure of the structure was not for the insufficient geometric stiffness of the column, but because of the large shear of the first support member below the top of the column. The H-shaped steel web was relatively weak and lack of shear stiffness. It reaches yield to failure under the complex stress state of axial compressive stress and shear stress. This failure mode was called the yield failure on the support of web.

4. Structural response characteristics for different eccentricity ratio

The response characteristics of H-section steel sections with H294×200×8×12, $b=1120$, $t=6$ and $l_0=3100/\text{mm}$ were analyzed. When the load reaches the limit value, the axial force value was denoted as N_{cr} , and the limit value of the bending moment was M_{1cr} ($M_{1cr}=0.078q_{cr}l_0^2$, q_{cr} was the limit value of transversal uniform load). The relationship curves of relative axial bearing capacity N_{cr}/N_y and relative moment bearing capacity M_{1cr}/M_y were shown as Figure 5. The bearing capacity of the column bore axial pressure independently was N_{ry} and the bearing capac-



Fig. 3. Buckling mode of the column



Fig. 4. Buckling mode of the column

ity under transversal uniform load was M_{1r} . The curve shape was convex, which indicates that when the column was under interaction of compression and bending load, $N_{cr}/N_{ry} + M_{1cr}/M_{1r} > 1$. To some extent, the bearing capacity of interactional compression and bending load was greater than that of axial force and transversal uniform load applied respectively. The reason was that, under the axial force, since wallboard shares part of loads, the axial pressure on column section along the height direction of the column attenuates quickly. The region of the maximum response of column was inferior of the top. Under the transversal uniform load, the maximum response of column was the first braced section of the mid-span region. When the two loads interacted, the position of maximum response was different. The maximum response caused by individuals cannot be superimposed. That was equivalent to weakening the maximum response caused by one of the loads. Consequently, the value of the $N_{cr}/N_{ry} + M_{1cr}/M_{1r}$ would be greater than 1.

The maximum lateral flexural-torsional deformation of the column generally occurs on the left limb. In different e values, the maximum X-direction deformation on the column when the load reaches limit point was selected, and the load-X displacement curve was shown in Figure 6. The curve shows that the stiffness of the column in different eccentricity was basically equal at the beginning of loading, and the lat-

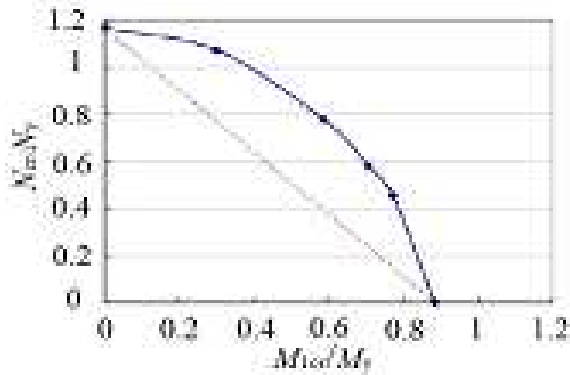


Fig. 5. Relationship curve between N_{cr}/N_y and M_{1cr}/M_y

eral deformation increases linearly with the loads. Approaching the limit load, the buckling occurs, the lateral deformation was accelerated to increase and the stiffness decreases significantly. After the limit point, the lateral deformation was still increasing rapidly while the load decreases, and the post-buckling performance of the structure was unstable. When the limit load was reached, the maximum X -direction deformation value on the column increased first and then decreased with the increase of e value, and there was little difference.

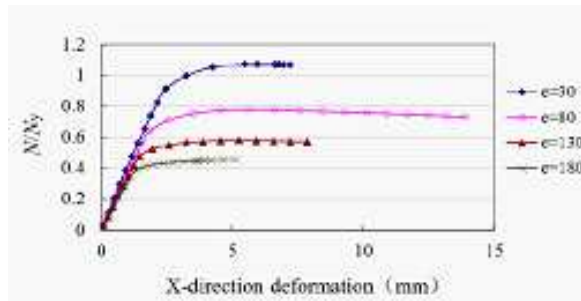


Fig. 6. Relationship curve between load and maximum column X -direction deformation

In different e value, the distribution curve of the section torsional angle along the height of the column was shown in Figure 7. As a smaller e value ($e=30$), axial force dominant in two loads. There were two flexural-torsional buckling occurring, one in the area below the column top ($0.9l_0$ section) under axial force, and the other in the mid-span area ($0.6l_0$ section) under the interaction of axial force and bending moment. Therefore, two torsional angles along the height direction were larger. And there were two buckling half-waves on the column. When e value was large ($e>30$), the control effect of axial force was weakened. Even at the limit load condition, the axial force of the column top area won't cause the flexural-torsional buckling on the front flange. Therefore, only the region of the mid-span occurs flexural-torsional buckling under the interaction of axial force and bending moment,

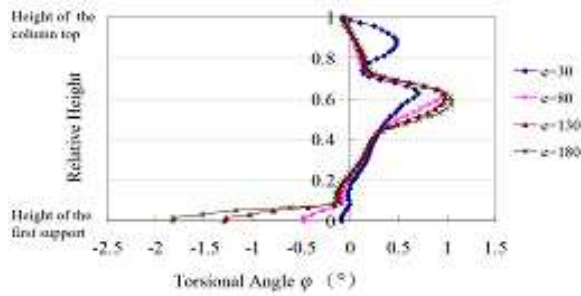


Fig. 7. Distribution of section torsional angle of column along height direction

where there was only one buckling half-wave on the column and the section torsional angle was significantly larger. With the increasing of e value, the section torsional angle of the mid-span increased when it reached the limit point. One side of the front flange in the first braced section below the column top was constrained by transversal connecting channels and the other side was free, where there could be torsion, and the torsional angle increases significantly with the increasing of e value.

5. Conclusion

The geometrical configuration and size of the examined wall-column structural system in precipitator casing was based on practical projects. Considering the influence of the structure initial geometric imperfections, the nonlinear finite element analysis of the load-bearing performance of the combined casing double limbs section column under axial and transverse load was investigated. The results indicate that there were three possible failure modes of the column. The first was the elasto-plastic flexural-torsional buckling occurring on the front flange closely below the top. The second was the elasto-plastic flexural-torsional buckling occurring on the front flange at the mid-span of the top column segment. The third was the strength failure occurring on the web at the first braced location where the stresses reached yielding because of significant not only compressive but also shear stresses. Buckling failure mainly occurred when axial compression was in high level. Strength failure mainly occurred when the bending moment was in high level.

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