STUDY ON LIMITED DUCTILITY CONNECTIONS WITH REDUCED BEAM SECTION UNDER CYCLIC LOADING

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Abstract— **Earthquakes in early 1980's fired up the studies on beam column joints. The welded connection was responsible for premature brittle failure of the joints. Relocating the formation of plastic hinge away from the critical section was taken as the prime objective by the researchers. In the present study asymmetric bolted beam column joints are considered. The main objective of this study is to enhance the seismic performance of existing buildings in low seismic region by reducing the beam sections. The post peak behaviour of the beam column joint is studied in detail. Comparing reduced web section and non-perforated section, comparing reduced flange section and non-perforated section, influence of web perforation size and effect of distance of web opening from column face are the parameter considered for this study. The study is conducted using Ansys Workbench 2022 R2 software. Contrary to the distance from the column face, the variation of the opening diameter has the greatest effect on the connection's strength and deformation. Reduced web section has 12% higher moment capacity than the reduced flange section. The plastic region can be moved away from the column face and into the beam extremely well by using big holes (0.9 h_b). The reduction of beam section as a bonus reduces the overall weight of the building.**

Keywords*— *Reduced beam section, dog bone, seismic behaviour, rotational ductility, ANSYS Workbench 2022 R2

I. INTRODUCTION

The use of reduced beam moment resisting bolted connections became widespread after the earthquakes in Northridge and Kobe. This chapter gives an overall idea of how reduced sections became popular, their benefits and their functionality. The mode of failure of reduced beam sections is discussed. The advantage of bolted connection over a welded connection is also looked into.

A. Northridge And Kobe Earthquakes

The Northridge earthquake in California in 1994 and the Kobe earthquake in Japan in 1995 brought to light how susceptible modern cities are to the consequences of natural disasters. Both occasions produced novel and unexpected results. They can be included among the significant metropolitan quake disasters of this century, which include Tangshan in 1976, San Francisco in 1906, and Tokyo in 1923 [1].

Compared to practically every other earthquake, the loss pattern and the age of the included structures were far more closely related in the Kobe earthquake. The years 1971 and 1981 are two significant dates in this regard for Japan. The Japanese earthquake code included the ductility principle later that year. In the Kobe commercial district, many of the older medium-sized buildings responded with brittle cracks, causing parts or occasionally entire floors of the structures to collapse. In the Kansai district, older structures were typically built to endure 20% of gravitational acceleration. The values measured in Kobe were up to 400% higher than those used in the original design. The geographical distribution of losses shows that even older structures functioned well in areas (like Osaka) where design values were not, or were only slightly, exceeded. Only when evident margins of design parameters were exceeded the differences between the behaviour of older and more modern structures were apparent. Earthquakes in Kobe and Northridge have shown that an important issue is the behaviour of older structures that no longer follow the most recent construction rules. Especially in those nations with sophisticated seismology and technology, this element is of utmost significance. Building earthquake-resistant structures is mostly done in these nations to protect people's lives and property..

The Northridge earthquake, in particular, demonstrated that the requirements for welded beam-column connections were insufficient to prevent connection failure or damage. Mainly, the failure in beam-column welded connection is a premature brittle failure [2]. Initially, the failure is thought to be due to poor workmanship. Later it was found that the assumptions were wrong. Investigations on beam-column bolted connections, plastic hinge formation, reduced beam sections etc. got accelerated after these earthquakes. Efforts were also made to make non-seismically designed existing buildings compatible with low-intensity earthquakes.

B. Plastic Hinge

The plastic limit analysis assumes that a rapid transition from elastic to ideally plastic behaviour occurs at a particular value of a moment, known as the plastic moment, denoted by Mp. When the plastic moment is reached, a plastic hinge is produced in the member. The hypothesis states that a plastic hinge, as opposed to a frictionless hinge, permits enormous rotations to take place at a constant plastic moment. A short beam is encircled by plastic hinges. Depending on the cross-sections and load distributions, these lengths' actual values can vary. However, thorough analysis has revealed that it suffices to treat beams as rigid-plastic, with plasticity restricted to plastic hinges in specific areas. Even if this assumption is adequate for limit state analysis, the spread of plasticity along plastic hinge lengths can be taken into consideration using finite element formulas. It is possible to create a kinematic mechanism that allows the system to move without restriction by incorporating a plastic hinge at a plastic limit load into a statically determined beam. It's referred to as the collapse mechanism. Every level of the beam's static indeterminacy requires the attachment of an additional plastic hinge to construct a collapse mechanism. With the help of reduced beam sections, the plastic hinge is introduced at a predetermined location to avoid brittle failure of the structures. Perfect incorporation of the plastic hinge and its performance saves human lives and property. Intentionally, the location of the failure is shifted towards an insensitive zone from the critical zone.

C. Vierendeel Mechanism

In perforated steel beams with a single big web aperture, when global shear pressures and Vierendeel moments coexist, the Vierendeel mechanism is always crucial. High shear forces exerted on the beam are linked to this mode of failure. By forming plastic hinges at the corners of the web opening forms and/or at specific locations, the tee parts above the web holes are stretched into a stretched shape. The first reports of this kind of failure for cellular beams and castellated beams were made in 1968 and 1957, respectively [3]. The tee sections above and below the web apertures of a perforated steel beam must support the applied shear as well as the primary and secondary moments. Shear force acting in the tee-sections along the horizontal length of the web opening causes the secondary moment, also known as the Vierendeel moment, to be created. The primary moment is the convectional bending moment. Therefore, the secondary moment is directly influenced by the web opening's horizontal length. Perforated beams with typical web openings have two fundamental mechanisms of collapse in the absence of local or global instability, depending on the shape and location of the web opening.

1) The top and bottom tee components of the tee have plastic tension and compression stress blocks in areas of considerable overall buckling.

2) The creation of plastic hinges at the four corners or at specific angles around the web opening causes parallelogram or Vierendeel motion in regions of high shear.

A single (isolated) web opening introduces three distinct modes of failure at the

perforated beams, according to a more analytical approach.

1) Due to a decreased moment capacity, flexural failure.

2) Lower shear capability leading to shear failure.

3) Vierendeel action is supported by the Vierendeel mechanism [3].

D. Reduced Sections

The beam flange is selectively trimmed in the vicinity of the beam-to-column connection in a reduced flange section moment connection, also referred to as the "dog bone”. The reduced flange section (RFS) restricts the amount of moment that may build at the face of the column and compels yielding and hinge development to take place inside the restricted portion of the beam. The reduced flange lessens the risk of fractures occurring in the beam flange groove welds and the surrounding base metal regions by minimizing demands on these areas. Although the reduced flange section effectively weakens the beam, it often has little effect on a steel moment frame's overall lateral strength and stiffness. Its main goal is to greatly increase ductility.

The beam web is cut out accordingly to induce a plastic hinge, in the case of reduced web sections (RWS). Inelastic behaviour in improperly built RWS beams is initially caused by a Vierendeel mechanism, in which local yielding occurs on the borders of the perforations. A connection can be made without significantly altering the beam web because the Vierendeel mechanism caps the shear that transmission from the beam to the beam-column. Additionally, panels between flanges and webs never deform coherently due to perforations, which reduces buckling and torsion.

E. Bolted / Welded Connections

The material used in welded joints lacks the holes required for bolted joints, they are often stronger than fastened joints. Bolted joints are simple, but welded joints have stronger strength, hence the manufacturing procedure is what ultimately determines joint strength. Welded joints are more durable than bolted joints because the cross-section is continuous. In contrast, the flexibility of bolted joints is increased by the displacement of the plates or angles that connect them during the transmission of stress. Therefore, bolted joints allow for more movement while exerting less pressure on the structure. The moment loading on the welded connections is insufficient, and they frequently experience local buckling, which compromises the structural stability. Because the weld is inflexible, progressive yielding cannot occur; instead, local buckling failure results when the local zone approaches the brittle zone. Therefore, bolted joints must be used in place of welded ones. Pressure vessels can be disassembled and moved to the bolted endplate connections, which also make them resistant to local buckling.

Reduced beam sections such as both reduced web sections (RWS) and reduced flange sections (RFS) typically weaken the beams and induce the plastic hinge at predetermined locations. The use of these sections comes in handy in regions where seismic activity is low. An extensive literature survey is dealt with, various experiments and investigations on reduced beam sections of various forms under various conditions.

II. NUMERICAL MODELLING

The finite element modelling of the bolted moment-resistant frame is done using commercial finite element analysis software Ansys Workbench 2022 R2. The finite element model is validated using the results from the experimental study by Tsavdaridis. et al., (2021)[4].

A. Geometrical modelling

The dimensions of the moment-resistant frame are given in Table 4.1. Fig 4.1 gives the side view dimensions of the moment-resistant frame.

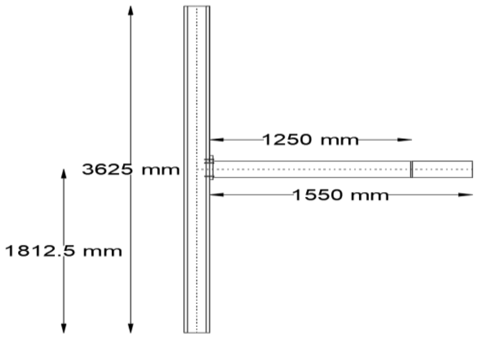

Fig. 1. Model Dimensions

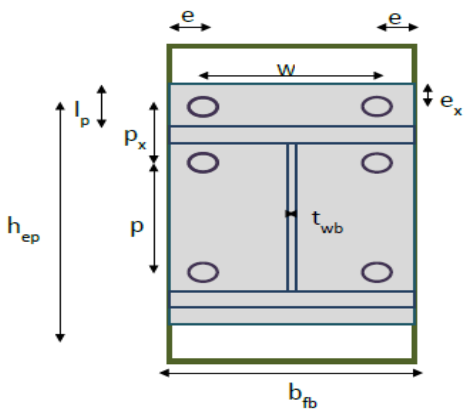

Fig. 2. Cross-Section Geometry of Side View

TABLE I DIMENSIONS OF MODEL

Geometric parameters	Dimension (mm)
e	30mm
e_x	30mm
p	20mm
p_x	90mm
w	90mm
b_{fb}	150 mm
h_b	298.9 mm
l_b	3100 mm
t_{fb}	10.8 mm
t_{wb}	7.3 mm
b_{fc}	160.1 mm
h_c	162.5 mm
H	3625 mm
tf_b	12.6 mm

B. Material modelling

A tri-linear stress-strain curve was used to depict material non-linearities. Fig. 3. depicts the tri-linear stress-strain model that was used to analyze.

TABLE I. MATERIAL PROPERTIES

Properties	Value
Young's modulus	210000 MPa
Poisson's ratio	0.3
Yield strength Of Column	289.4 MPa
Yield strength Of Beam	308.5 MPa
Yield strength Of End Plate	291.5 MPa
Yield strength Of Bolt	900 MPa

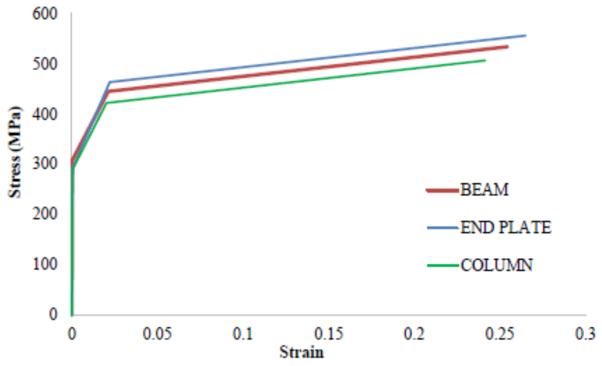

Fig. 3. Stress-strain curve [1]

C. Boundary Condition And Loading

The boundary conditions are applied on the top and bottom faces of the column. The top is constrained in the x-axis direction with movement allowed in the y-axis direction as in Table II and Fig. 3. The bottom is constrained in the y-axis and z-axis directions with movement allowed in the x-axis direction as in Table II and Fig. 3. Loading Sequence FEMA 350 is adopted for this FEM analysis Fig. 4.

TABLE II BOUNDARY CONDITIONS

Boundary condition	U_x	U_y	U_z	R_x	R_y	R_z
Top	0	Free	0	0	0	0
Bottom	0	0	0	0	0	0
Loading plate	0	Cyclic	0	Free	0	0

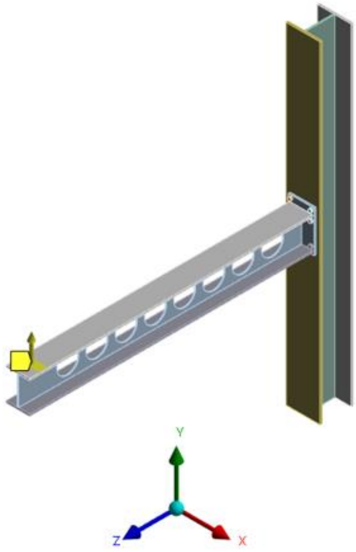

Fig. 3. Stress-strain Curve

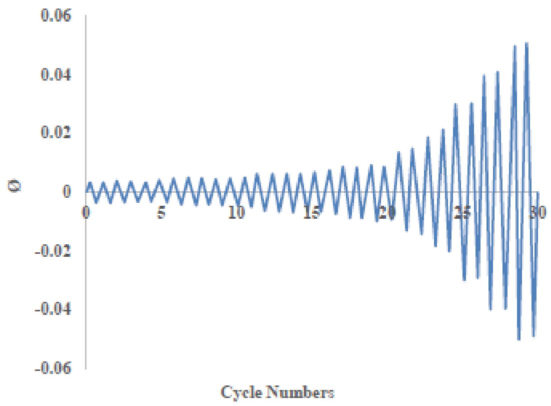

Fig. 4. FEMA Loading Protocol

D. Validation

The finite element model is validated using the results from the experimental study by Tsavdaridis. et al., (2021)[4]. The perforated model named Specimen - 7 and the non-perforated model named solid model was considered for validation. Skeleton curves of both models were taken for comparison as shown in Fig. 5 and Fig. 6.

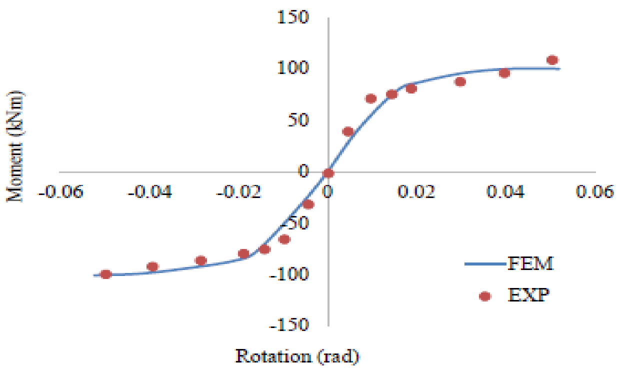

Fig.5 Comparison of FE and Experimental Skeleton Curves-Perforated Model

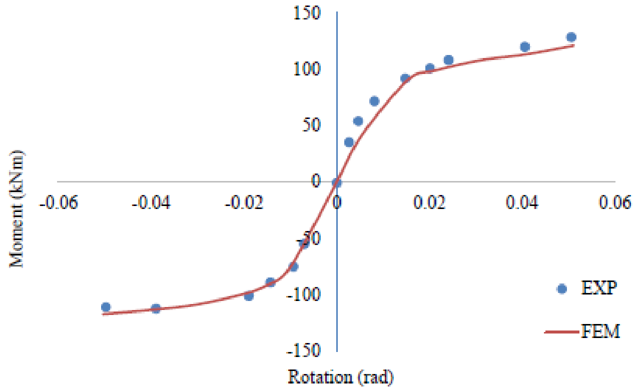

Fig. 6 Comparison of FE and Experimental Skeleton Curves – Solid Model

Failure mode of the validated specimens indicates that column also fails along with the beam as in Fig. 7

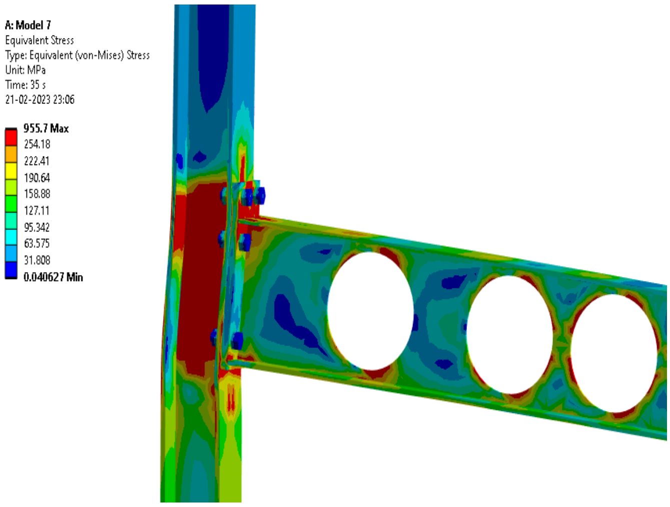

Fig. 7 Failure Mode of Perforated Model

E. Design of Beam and Column

Column and Beam sections were taken with the help of the AISC steel construction manual 14th Edition. The flexural capacity of the column is taken as 1.5 times the flexural capacity of the beam (In accordance with the 'weak beam–strong column' mechanism). The adopted Beam and Column sections are W 14 x 68 and W 14 x 109. All other properties of the model are taken as per the validated model.

III. PARAMETRIC STUDY

The parametric study was conducted by using the designed model. The length of the column and beam taken the same as that of the validated model is 3.625m and 3.100m respectively. The length of loading from the centre of the column is 3.152 m. According to FEMA 356, the maximum allowable connection rotational limit is 0.05 rad for reduced beam sections. Dimensions of models used for parametric studies are explained in respective sub-sections.

A. *Influence of Reduced Beam*

The web-perforated and flange-perforated models are compared with the non-perforated model. Only single circular perforation was considered for this comparison. The diameter of the hole is taken as 0.9 h_b and the distance of the hole from the column face is chosen as 1.2 times beam web height. The flange cut is also provided at a distance of 1.2 times from the column height. The skeleton curve of web perforation and flange cut are separately compared with no perforation as in Fig. 8 and Fig. 9. The results obtained are shown in Table III and Table IV. The reduced web section has a 12% higher moment capacity than the reduced flange section. The rotational ductility of the reduced web section is 25.89 % lower compared with the reduced flange section. Yield rotation is increased for flange cut and is decreased for web cut when compared with the non-perforated specimen.

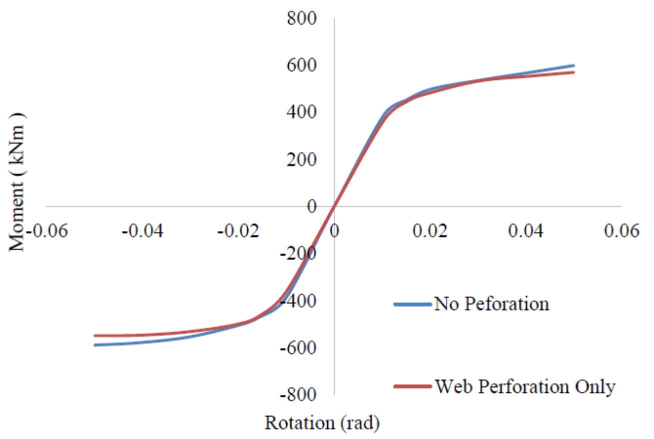

Fig.8 Skeleton Curve of No Perforation and Web Perforation Only.

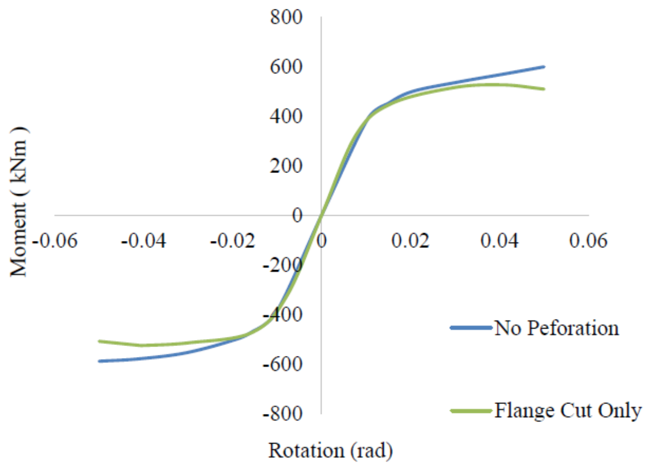

Fig.8 Skeleton Curve of No Perforation and Flange Cut Only

TABLE III Results of No Perforation and Web Perforation Only.

Model	Yeild Moment (kNm)	Ultimate Moment (kNm)	Rotational Ductility	Initial Rotational Stiffness (kNm.rad)	Total Energy Dissipated (kNm.rad)
No Perforation	400	598	4.854	46667	155.388
Web Perforation only	366	569	4.812	42577	150.692

TABLE IV Results of No Perforation and Flange Cut Only

Model	Yeild Moment (kNm)	Ultimate Moment (kNm)	Rotational Ductility	Initial Rotational Stiffness (kNm.rad)	Total Energy Dissipated (kNm.rad)
No Perforation	400	598	4.854	46667	155.388
Flange cut only	322	508	6.493	32609	147.475

B. *Influence of Web Perforation Size*

For finding out the influence of web perforation size the, the number of web openings was kept as constant. The distance between the centre to centre of the holes and distance of the hole from the column face are kept as 1.2 times the beam web height. The diameter of holes adopted for this parametric study are listed in Table V. Skeleton curves are shown in Fig. 9. From Table V it is observed that the yield rotation increases as the perforation size increases and rotational ductility has got only negligible influence as the perforation size is varied (only 2-3 % variation).

TABLE V Diameter of Holes for Influence of Web Perforation Size

Model	Diameter of hole (mm)
0.6 h_b	191.41
0.7 h_b	223.32
0.8 h_b	255.22
0.9 h_b	287.12

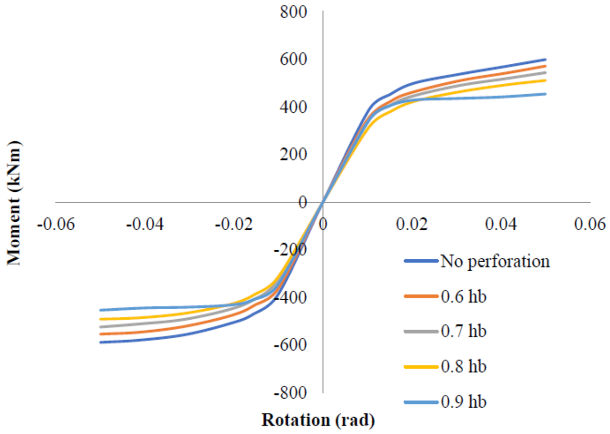

Fig. 9 Skeleton curve – Web Perforation Size

C. *Effect of Distance of Web Opening From Column Face*

Centre to centre distance between the holes is kept as 1.2 times the beam web height for studying the impact of the web opening's distance from the column face. Diameter of the hole is kept as 0.9 h_b. Number of holes adopted is two for this study. Skeleton curve for this study is shown in Fig. 10. Table VI shows the results extracted from the curves.

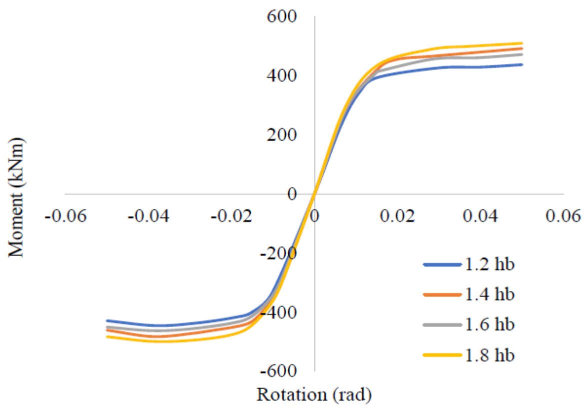

Fig. 10 Skeleton Curve - Effect of Distance of Web Opening from Column Face

TABLE VI Results of Effect of Distance of Web Opening from Column Face

Model	Yeild Moment (kNm)	Ultimate Moment (kNm)	Rotational Ductility	Total Energy Dissipated (kNm.rad)
1.2	232	437	7.81	138.21
1.4	281	491	6.78	148.23
1.6	293	471	6.45	143.85
1.8	291	509	6.78	150.91

IV. CONCLUSIONS

- Contrary to the distance from the column face, the modification of the aperture diameter has a greater impact on the strength and deformation of the connection.
- Reduced web section has a 12% higher moment capacity than the reduced flange section.
- Rotational ductility of the reduced web section is 25.89 % lower compared with the reduced flange section.
- The plastic region can be moved away from the column face and into the beam extremely well by using big holes (0.9 h_b)
- Similar to connections made using non-perforated beams, small apertures caused strains to build on the face of the column.

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