them. These briquettes were four and a half months to twelve months old, and all showed a loss of strength due to the heating, the loss ranging from 58 per cent. to 89 per cent. Mr. Shankland.

In reply to Mr. Max am Ende, the fibre stress in the columns never exceeded 15,000 lbs. per square inch for live and dead load, and was further reduced for long columns. It was improbable that the 25,000 lbs. fibre stress spoken of as resulting from the combined live and dead loads, and a wind-pressure of 30 lbs. per square foot, ever obtained in a column; if so, it occurred only at long intervals and was of momentary duration. As stated in the Paper, this was not more than in the best bridge practice, which allowed the combined stress from live, dead and wind loads to be as high as 25,000 lbs. per square inch. That amount was allowed as a maximnm in a heavy railway bridge of 525 feet span built a few years ago.

Correspondence.

Mr. S. G. Artingstall observed that Chicago lay on the edge of an ancient lake, of which Lake Michigan was the existing evidence. The lake had as an outlet a river draining towards the Gulf of Mexico by way of the Desplaines Valley, the Illinois Valley, and the Mississippi River. Chicago was situated over the bed of that ancient river, and the character of the deposits upon which the city was built and upon which the buildings were erected was uncertain and unreliable. The business part of the city was on the southern part of the main channel of the ancient river, the depth to the rock being 80 feet to 100 feet below city datum, or 94 feet to 114 feet below the level of the streets. This rock was overlaid with a cemented gravel or hard pan, over which was clay, tolerably firm at first, but quickly changing to a soft clay of about the consistency of soft putty. Above the sewers, or about the level of the water in the lake or river, the ground water had been drained off and had left a firm soil or deposit, upon which ordinary buildings could be erected with security. Mr. Artingstall.

When heavy structures had to be erected, the engineers had found out by experience that ordinary formulas could not be depended on, but that the area of the foundations under each wall, pier, or column must be carefully proportioned to support its part of the load, or unequal settlement would result. That had been shown by numbers of buildings, as for instance, the Board of Trade. The tower, about 300 feet high, had been taken down

Mr. Artingstall. because of its badly proportioned foundations. The Dearborn Station tower had been cut away from the main building twice, so as to allow it to settle and not wreck the building. In the City Hall, the floors in a span of 40 feet were 18 inches to 24 inches out of level before the building was completed, and the floors and ceilings are furred to that extent to make them level. Many instances could be given where foundations were built in the usual ordinary manner, and disaster to the structures followed, but by experience the foundations of all important buildings received great attention and careful design.

In constructing the tunnel under the river at Harrison Street, the ground (clay) was so soft, and the use of compressed air not being available, that all the miners put in the face could not remove it as fast as it flowed in. Again, in dredging the river and harbour, of which Mr. Artingstall had charge for more than twenty years, when a channel of 18 feet to 20 feet in depth was made, it was found that one or two years afterwards it was but 14 feet to 16 feet deep. The next dredging showed very little deposit on the surface, but always virgin clay, indicating clearly that the natural strata of material which underlay the central or business part of the city was of very soft or semi-fluid material.

Mr. Howe. Mr. HENRY M. HOWE confessed some apprehension as to the resistance of the steel structures to corrosion and fire. As the strength of a chain was measured by its weakest link, so the strength of any part of a steel skeleton structure was that of the most rapidly corroding member below it; and its safety in regard to fire was that of its member most exposed to the heat, in the sense that the failure of a lower member, whether from corrosion or collapsing, might bring down the whole structure above it. The rate of corrosion varied so very greatly with the conditions, that any formula, like that quoted in the Paper, should be used only with great caution. The amount of salt, of moisture, of sulphurous acid, and perhaps of ozone in the air, the temperature, the wind, the thoroughness of protection by paint, &c., all affected the corrosion greatly. It appeared to him yet to be shown that those in charge of the tall steel structures would, or could, invariably exercise such supervision and so protect the metal from corrosion as to guarantee its safety. It would be difficult to exaggerate the care that should be taken to protect from corrosion all the steel parts, especially those which must be inaccessible for inspection, for renewal of protective coating, and for replacement. While fires within common office buildings were unlikely to heat the steel skeleton so much as to cause it to buckle, yet fires from without might well do so. A steel skeleton building might easily be menaced by the neighbourhood of inflammable buildings. Mr. Howe could readily understand that, if but a moderate thickness of brickwork or stone were between the steel skeleton and the outer face, the burning of neighbouring buildings might readily heat that skeleton to such a degree as to cause collapse. As that would be most likely to occur in the lower storeys, its consequences would be terrible. It might be well that the steel skeleton of the outer walls either should begin far above the ground-level, or should be protected against fire by an ample thickness of brick or of stone. Against earthquakes the steel skeleton construction should give much greater security than masonry of equal height. Mr. Howe.

Mr. W. L. B. JENNEY mentioned that in the autumn of 1892 he was appointed architect for a tall fireproof building, which the Home Insurance Company of New York proposed to erect in Chicago. The instructions were that all storeys above the bank-floor should be divided into the maximum number of small offices, all well lighted. As was foreseen, this arrangement reduced the piers between the windows, so that the ordinary masonry construction would not carry the load in lower storeys. To thicken the piers into the building was objectionable. The natural solution was to build an iron column in each pier to carry the floors. The question of expansion and contraction of continuous columns 150 feet high subjected to a variation of some 120° F. then presented itself. This suggested carrying the walls as well as the floors, storey by storey, on the columns, thus dividing the movement. The details followed naturally. That system of construction resulted in another very decided advantage when building on a compressible soil, such as underlay the business district of Chicago. The outside walls, being only self-sustaining and of the height of a single storey, could be reduced to the thickness necessary to fireproof the steel, to hold the window-frames and to produce the desired architectural effect, greatly reducing the load on the foundations. Mr. Jenney.

It was the result of experience that the clay underlying Chicago under a given load not exceeding, say, 4,000 lbs. per square foot compressed to a certain point, soon stopped and was not further compressed without a material addition to the load. After the clay had been compressed for a considerable period certain additions could be made to the load without further settlement. The settlement increased rapidly as the load increased, after passing the 3,500 lbs. limit. Mr. Jenney's experience had been that if the maximum load on the clay did not exceed 3,500 lbs. per square

Mr. Jenney. foot for buildings not exceeding 160 feet in height (unless of very large ground area) the settlement would not exceed 4 inches. As an illustration of the principle that light loads would only produce small settlements, the Fair, a store building, Dearborn and State Streets, Chicago, was calculated for sixteen storeys. Only nine were built. The levels taken four years after showed a maximum settlement of only $1\frac{3}{8}$ inch. As stated by the Author, the practice had been not to cut into hard pan. This so-called hard pan differed from the clay for many feet below only in containing less water. The value of the hard pan was that it added something to the shearing on the outline of the foundations as they settled into the clay. Certainly this was not very important. In the Young Men's Christian Association building, commenced 1892, the necessity for a high basement to accommodate a swimming tank, boilers, etc., forced Mr. Jenney to cut some 2 feet into the hard pan. The actual load on the clay was 3,500 lbs. per square foot. The maximum settlement from levels taken 10th January, 1894, was $2\frac{1}{16}$ inches. The building was erected much more slowly than any other of the Chicago tall buildings, two years from commencement to finish, instead of about ten months, which no doubt reduced the total settlement. It was well known that in driving piles into Chicago clay, quickly repeated blows were the most efficient, and that if a pile was partly driven and then left, say, one night, materially more force was required to start it again than if the driving had been continuous.

The calculation of the foundation beams has been a matter of considerable discussion among American engineers. The method described by the Author was extensively used, and practical results seem to justify it. Another method, which was used by Mr. Jenney, was to consider each layer of beams separately, and find the moment at the centre; the concrete was not considered as subjected to horizontal stresses. By this method more steel was required, which was not objectionable, as the steel in the footings might be subjected to conditions likely to produce corrosion. For this reason it was his practice to have the foundation beams, as well as all other steel in the building, oiled at the mill and painted at the building. As all assembling was done with hot rivets most of the painting had to be done after erection.

There had been a sufficient number of opportunities to obtain the relative cost of the two systems of construction to establish the fact that the steel skeleton was the least expensive. The steel skeleton had been in use some thirteen years, and had been subjected to all the usual conditions. It had been shown by tests made during a severe gale that it was even more rigid than heavy masonry buildings. A cyclone might blow in the windows, but the frame being a network of steel, braced in every direction and securely riveted together, would sustain less damage than other systems of construction; so that it might be truly said of these buildings that they were wind-proof, fire-proof, and earthquake-proof. Mr. Jenney.

Mr. G. S. MORISON remarked that the Paper described a new system of construction which was remarkable for its rapid development. The rapid development of any novel construction was sure to be accompanied by defects whose importance was not at first realized; and he considered that the system of skeleton construction was no exception to the rule. Iron bridge building in America might be given as an instance. The earliest bridges, generally designed by careful engineers, were followed by a class of structures built by designers who studied simplicity and economy of construction. Those structures fulfilled the requirements for vertical loads, were economical in material and were erected with extraordinary rapidity; but they were loosely put together and were adapted so closely to existing rolling-stock that they were quickly superseded. Constructed by contract from contractors' designs, they were still used largely for highway work; but on railways they had generally been superseded by carefully designed structures, in which something more than the weight to be carried was considered, and which were equal in rigidity and massiveness to the structures built in Europe. Steel skeleton construction was in its earlier stage. The novelty came with the first design; the economy was at once apparent; buildings in which everything was reduced to a minimum of material represented the present state of the art. This construction was now being used largely in New York; but as the natural foundations were much better there than in Chicago, and the general demand was for better structures, although some exceedingly bad work had been done in New York, the better class of buildings were better built there than in Chicago. The buildings in the latter city were adequate to carry the weights which were likely to be imposed upon them, though the stresses and weights given in the Paper showed that the factor of safety was not large; while the formula used for the strength of long columns was one not properly applicable to any but comparatively short columns. Settlement occurred in the foundations, and that had produced, even in some of the later buildings, deformations which though perhaps not serious were at least displeasing. Vibrations from machinery or passing heavy Mr. Morison.

Mr. Morison. loads were distinctly felt in the highest storeys. The provision to resist wind-pressure was often imperfect.

The general method of spreading foundations over a considerable area of soft material was not new, though it had seldom been found absolutely satisfactory; it was, however, cheap and quickly done. In structures of irregular shape, in which very different weights were carried by different columns, it did not give good results. If the areas of the footings were proportioned according to the weight carried, uniform pressures were given at the bottom of the footings only; but the compression of the soft material extended indefinitely below those footings, and the pressure ceased to be uniform lower down. For that reason, light columns with small footings generally settled less than the heavy columns. That system of foundation could be made really satisfactory only for structures of regular, uniform shape, in which the weights of the several columns were kept uniform, so that footings of equal size were used. The general principle of all those designs was to provide a metallic skeleton to carry the weight. That was perfectly simple. The weight of each separate storey, including the outer walls, would be transferred directly to the skeleton frame, and the modulus of elasticity of ordinary masonry was so small in proportion to that of iron or steel that it readily accommodated itself to any compression caused by weight. The same might be said of temperature. The expansion of masonry and steel was so nearly equal, that when they were kept at the same temperature no disturbance need be expected from that source, and the enclosure of the steel skeleton in some form of masonry accomplished that result. When, however, other elements were introduced, the conditions were different. The rigidity of such a structure was elastic, and not the rigidity of mass. A tall, narrow building was subjected by wind to a horizontal pressure which it could only resist as a vertical beam. If such a beam had strength to resist the maximum wind-pressure, the building as a whole was safe. If the bending was insufficient to crack the masonry, the building was all right. As no one knew the maximum possible wind-pressure, it was wise to utilize that form of construction which would give the least deflection with permissible stresses, and that was the one in which the members relied upon to resist bending had the greatest horizontal dimensions.

The Paper described three methods of resisting wind-pressure. The first was by a series of diagonal rods, which really formed a system of trusses whose depth was the width of the building.

That, with proper details, was the best method of the three; but the Author had discarded it for no better reason than that it might interfere with partitions. The second method was by the use of portals connecting the vertical posts—a good arrangement if carefully worked out, though it necessarily threw some bending stresses on the posts; but that was rejected because of its cost. The knee-braces mentioned in the Paper were only a modified form of portal bracing. The third method relied entirely upon the stiffness of the posts and their connections to resist lateral flexure, thus reducing the width of the beam from that of the whole building to that of a single column. Such reduction was, however, less important than might at first sight appear, because the length of each separate member was reduced to that of a single storey. It had been adopted generally because of its convenience in construction. It was safe if carefully carried out; but it greatly increased the stresses to be resisted by the posts and made the design of the posts of special importance. Mr. Morison.

There was another method of resisting wind-pressure, to which the Author made no reference; but which, in Mr. Morison's opinion, was better than any of those named in the Paper. It was used in most of the earlier buildings. It consisted of making the outside walls of the building composite beams, in which the vertical posts formed the chords, or booms, of the trusses, and the floor-beams combined with the brick walls formed the webs. A building of that kind enclosed by four walls, with proper details for metallic connections and for the support of the filling, and with windows and openings carefully placed, was a very satisfactory structure. It was really a box composed of four deep vertical beams, stiffened by as many horizontal beams as there were floors. He considered that when the subject had been fully investigated that would be the only form of skeleton structure permitted. The outside walls, with such help as they might get from partitions, were the main-stay of some of the buildings. While a building with four walls was decidedly the best form, a satisfactory structure might be made with only two walls provided those two formed adjacent and not opposite sides. If, however, the "table-leg" principle, which relied only on the strength and stiffness of the columns to resist wind-pressure, was used, it was of the utmost importance that the connections between the floors and the posts should be as rigid as possible, and that the posts should be properly designed to act as vertical beams.

The designs described in the Paper did not meet these require-

Mr. Morison. ments, and strength and durability were sacrificed for speed and convenience of erection. The floor system should be carefully riveted to the posts, so that there would be no lost motion; but the specifications provided for bolts instead of rivets, and instead of being turned with a driving fit in reamed holes, these bolts were $\frac{1}{32}$ inch smaller than the holes. The objection advanced by the Author to box and Z-bar columns was that he thought them laterally weak, whereas both of those forms of column had continuous webs, and their strength and rigidity could be determined absolutely by well-known principles. It was only necessary to increase their dimensions until the necessary rigidity was obtained. The column, known in the market as the "Gray" column, which undoubtedly possessed great advantages in regard to rapid erection, was unfit to resist transverse strains, and the only tests which had been published indicated that it was rather weak when considered only as a post. This column was formed of eight angle-bars, which, in the lower storey of the Fisher building, appeared to be connected by three sets of tie-plates of thin metal 8 inches wide; such a column was really without a web. When subjected to bending-stress, it was not a single post but a bundle of eight faggots. Within small limits it might be safe; but its stiffness could only be secured by extreme local strain in the angles and extreme bearing strain in the outside rivets of the tie-plates. If continuous plates were substituted for the tie-plates so as to provide a web, that column might be an excellent one. As constructed, it was without either a radius of gyration or a moment of inertia, though the latter element had apparently been determined on the supposition that the tie-plates were equivalent to a web. In the Fisher building, which was referred to in the Paper as the highest development of this class of construction, the entire wind-stresses were resisted by those webless columns. Apparently, the strength of the columns as beams had been calculated on the basis referred to. The building stood without brick walls on any side, having only a covering of thin hollow terra-cotta blocks on the north side, the other three sides consisting of bay windows. The continued existence of the building would confirm the view held by many engineers, that a wind-pressure as high as 10 lbs. per square foot over the entire surface of a large building was of extremely rare occurrence.

Mr. Robson. Mr. J. J. Robson inquired the thickness of the cement and six-ply tar and gravel on the roofs. The gas- and water-pipes appeared to be conveniently hidden in hollow spaces in the columns; how were they rendered accessible in case of accident or renewals?

Mr. Robson.

What description of tiles was used for the fireproofing surrounding the columns? He considered the pile foundation bad in principle; the piles seemed to be unevenly loaded, and the outside piles would really bear a greater weight than those inside. The cantilever projection of 6 feet could have been reduced by one or two additional rows of piles, even if those close to the footings of the adjacent buildings had to be screwed instead of being driven. The cost given averaged about 1*s*. 10*d*. per cubic foot, which was twice the price of large hotels and blocks of flats recently erected in London (which averaged between 10*d*. and 1*s*. per cubic foot). His experience of ordinary building work in America was that it cost about the same as good work in England, but any special or artistic work cost very much more. An allowance for a wind-pressure of 30 lbs. per square foot appeared to be sufficient for America, but would not be safe in England. That allowance of American engineers confirmed his opinion that the wind velocities of America were not so great as those which were sometimes experienced on the coasts of Great Britain. The highest wind-velocities obtained in England were those recorded at Fleetwood on the Lancashire coast, where the Meteorological Office had a station. There gales frequently attained a velocity of 80 miles or 90 miles per hour, whilst during the great gale of 22nd December, 1894, the average for the twenty-four hours was 96 miles per hour (= 27·65 lbs. per square foot); the highest average over one hour was 107 miles (= 34·35 lbs. per square foot); whilst the highest average during twenty-three minutes was 123½ miles per hour (= 45·75 lbs. per square foot). He believed that in England it was necessary to allow for a minimum pressure of 50 lbs. per square foot, for in addition to the results mentioned from Fleetwood (which were averages), there were the momentary gusts which it was impossible to record.

Mr. Strobel.

Mr. Charles L. Strobel pointed out that the most important consideration in designing the steel skeleton for a tall building was the proportioning of the parts and their connections so as to afford sufficient resistance to wind-pressure. Some resistance would be afforded by the partition walls and the outside veneering of terra-cotta in buildings of the description referred to; but those parts were usually made so thin and light that the steel frame of the building was called on to furnish nearly all the resistance to wind. Further, for convenience during construction, all the storeys above the second or third floors, including the roof, were frequently completed while the first or the first and second storeys were still without outside walls or partitions, so that, as

Mr. Strobel. long as that condition lasted, the ironwork was called on to resist all the wind-stresses.

The most efficient and economical wind-bracing was a system of diagonal rods in vertical planes, making a braced tower of certain parts of the building; but that type of wind-bracing was frequently inadmissible on account of its interference with doors and windows. The portal bracing referred to by the Author had been used for very high buildings with small bases, and for such cases, if diagonal rods were not admissible, would seem to afford the most satisfactory solution of the problem. That plan not only provided for the considerable number of rivets necessary for the connection of girders to columns to resist the bending moments, which were very large in such cases, but the column itself was greatly strengthened for resisting transverse stress. For those conditions, the column shown in Fig. 3, Plate 1, consisting of eight angle-bars, connected by bent batten plates, would be inadequate, as not possessing sufficient bending strength, and for other reasons. That column had only the transverse strength of the eight angle-bars composing it, acting independently and not as a unit. Further, in the case of heavy columns, the number of rivets connecting one segment with another would be entirely inadequate to resist such shear as was likely to occur; and there was not sufficient strength locally, where the girder was connected with the column, to resist the couple acting at that point. The effect of the large horizontal forces acting at that point would be to press the two angle-bars of the column in at the lower flange of the girder, and to pull the two angle-bars at the upper flange of the girder out, or *vice versâ*, the batten plates resisting this action only by their resistance to bending, which was slight. For all cases requiring that the columns should furnish considerable resistance to bending, they should be made of continuous metal closely riveted.

The Author had not done full justice to the advantages of the use of joint-plates. The main reason for their use had been to assure good bearing between the abutting surfaces of two-columns sections, and to provide for carrying several beams alongside of one another, as was frequently desirable where a massive treatment of the walls was desired. For those reasons joint-plates were used, for instance, in the Century Building, St. Louis, at present approaching completion, where Georgia marble was used for the outside walls instead of terra-cotta, and the corner, and some other piers were built of solid masonry. The joint-plate admitted of an excellent connection between beam or girder and column for resisting wind-stress, as the rivets were strained in shear and acted at a maximum leverage. In the case of direct connection of girders with columns by angle-bars riveted to the web of the girder there was tension against the rivet-heads, and the rivets acted with an unequal and smaller leverage. The joint-plate should reach out so as to take hold of four or more rivets at the bottom of the beam or girder. The upper connection of the beam or girder was less convenient, but could be made entirely satisfactory. Usually angle-bar connections were used. One half of those connections would be strained in tension, so that a pull would be exerted upon the rivet-heads. The other half, however, were strained in compression; for those the girder could, in addition, be made to abut directly against the column by inserting iron wedges. Mr. Strobel.

Generally, it was not convenient to rely upon the interior columns furnishing the full resistance of their strength to wind-pressure. The connection between beam and column was then made of such strength as was convenient, so that a part only, say one-fourth, of the total wind-pressure was resisted by the interior columns. The remainder of the total wind-pressure was then resisted in the planes of the outside columns, either by a bracket-connection between the wall girders and the columns, or by making the girders sufficiently deep, so that by riveting direct to the column without brackets sufficient strength of connection was obtained.

Mr. Strobel considered it inadvisable to impose as high a stress as 15,000 lbs. per square inch for vertical loads upon columns, and 25,000 lbs. per square inch for combined vertical loads and wind-pressure. He had adopted 12,000 lbs. per square inch and 18,000 lbs. per square inch respectively as the limits, and in the early period of the construction of high buildings those limits were generally adhered to. It was true that stresses of 16,000 lbs. per square inch were customary for beams and girders in buildings, but those parts were not as likely to be overstrained from causes which could not be controlled as columns were, and the failure of a beam in a building was a much less serious matter than the failure of a column. The following considerations caused him to think that more moderate stresses should be used for columns in buildings. The settlement of buildings was rarely entirely uniform, and especially was that true for the conditions which prevailed in Chicago. Inequality in settlement would throw additional loads upon certain columns, relieving others, and might also cause distortions which would considerably modify the stresses obtained by calculation. Those inequalities of settlement in the case of one tall building in Chicago had caused it to be as much as 6 inches out of the perpendicular. Comparison had been made with the stresses

Mr. Strobel. customary for bridge work; the causes referred to, and which resulted in large secondary stresses, did not, however, exist in bridge work; and further, the care and thoroughness with which the latter class of construction was carried out was not usually possible in building work. One reason for this was that building work in America was usually hastily executed. Then again, it was difficult to get at some of the parts to perform the riveting efficiently, so much so, that bolts were frequently substituted for rivets in those connections, either in whole or in part, whereas allowance for the weakening effect of bolt-holes in columns was not ordinarily made in the calculations.

In conclusion, it should be stated that, so far, experience with completed buildings of the description considered would seem to indicate that a wind-pressure of 30 lbs. per square foot over the entire surface of a building was never even approximately realized in a compactly-built city, excepting, perhaps, in the case of tornadoes of the most unusual and violent kind, such as destroyed a part of the city of St. Louis in 1895. Tempests of that description would destroy all the glass windows in a building, and probably the terra-cotta veneering would be blown off as well, and the question of course arose whether a building should be designed to withstand forces of such unusual kind, which, for most localities, might be likened in the rarity of their occurrence to those arising from earthquakes.

Mr. Wilson. Mr. JOSEPH M. WILSON remarked that the method of steel skeleton construction had been a natural growth from the substitution of iron and steel for wood, and the demand for higher buildings which the new material rendered it possible to build. The original framed wooden building was essentially a skeleton construction covered outside with wooden boards. Then came the suggestion of a less inflammable material for this covering, and as long ago as 1873 he had suggested for temporary buildings of the Centennial Exhibition thin walls of brick as an outside covering, stiffened inside with timber framing, by which the roof was also to be carried. Residences were built in that way in many of the States, and he saw, several years ago, one under construction at Milwaukee with a skeleton of timber framing and a 4-inch facing of brick built outside anchored to the timber at frequent intervals, making in many respects a very good and serviceable building. In 1880–81 he had used, in the construction of the Broad Street Station, Philadelphia, wrought-iron columns from the ground floor upwards, encased in masonry carrying a second floor, then extending to the third floor and carrying plate-girders, on which was built the upper part of

Mr. Wilson.

the rear exterior wall of the building. The structure was subdivided into a series of parallelopipeds, wrought- and cast-iron columns being placed at the intersections on a similar system to that now adopted for skeleton buildings; and the interior walls and floors were carried on girders between the columns, the building being a type of the more perfect skeleton structures which came afterwards. In 1887, the Drexel building in Philadelphia was designed by Mr. Wilson's firm, and was built on a system of skeleton construction, columns being placed in the exterior walls, which carried their own weight for the full height, but were stiffened laterally by the encased columns which also carried the ends of the floor girders. The skeleton construction sustained the floors and division walls independently in each storey, thus allowing any arrangement of rooms and fireproof partitions, without reference to their disposition in the other storeys. Philadelphia was fortunate in having, as a rule, good foundations, generally of gravel, and the load placed on such foundations was usually between 6,000 lbs. and 8,000 lbs. per square foot. Some foundations, as those of the City Hall, were carried down to water-gravel, which consisted of a mixture of gravel and clay, in what might be called the first stage of rock-formation. He had seen a case where the load on that material was 12 tons per square foot, but some settlement had taken place in it.

Referring to the Author's practice in regard to wind-pressure, the maximum stress for live, dead, and wind-stresses, while allowable in Mr. Wilson's opinion for buildings, was beyond what he would use in bridges. In his practice he made it for bridges from 15,000 lbs. to 17,000 lbs. per square inch in tension members, and not more than 15,000 lbs. per square inch in compression members, with a reduction, of course, for length. He quite agreed with the Author as to the preservative properties of Portland cement, but it should be free from lime, otherwise it might expand and crack the steel when in a confined place, as had occurred in his experience.

Mr. Shankland.

Mr. SHANKLAND, in reply to the Correspondence, agreed that the method of calculation referred to by Mr. Jenney was theoretically correct for parallel tiers of beams piled loose; but the beams as actually placed in the foundations, or in layers at right-angles to one another, and embedded in a solid mass of concrete, made the whole foundation homogeneous, like a masonry wall. The following formula[1] had been given by Prof. Malverd A. Howe, for

[1] "Retaining Walls for Earth," Third Edition, 1896, p. 80.

Mr. Shankland. the safe projection for iron or steel beams embedded in concrete:— Let I denote the moment of inertia of the section; h, the depth of the beam; p_o, the intensity of the pressure upon the bed of the foundation transmitted to the beam; R, the modulus of rupture of the material composing the beam; and F, a factor of safety.

Then
$$p_o \frac{O^2}{2} = 2 \frac{R\,I}{F\,h}$$

or
$$O = 2 \sqrt{\frac{R\,I}{F\,p_o\,h}}\,I$$

Taking the foundation mentioned at p. 17, more beams would be required by the formula of Mr. Jenney, and less by Prof. Howe's formula, than were actually used, the latter number being about the mean of the two. The Table of safe projections for steel beams given in the "Pocket Companion" of the Carnegie Steel Company agreed with Professor Howe's formula. The formula for corrosion was only cited as a curiosity. The outside columns were, as stated, filled with concrete to resist corrosion and fire. It had been truly remarked by Mr. Howe that the greatest danger of fire came from adjoining buildings, and not from within. Every piece of the steel frame was carefully covered with fireproofing material, and when that was done and the outside columns were enclosed with brickwork and filled with concrete, it seemed that all possible precautions had been taken to protect the building both from fire and from corrosion. In reference to Mr. Morison's remarks, it must be observed that, in designing foundations for piers, proportioned to the load they had to carry, variations were always made in the weight allowed per square foot, depending upon the relative position of a footing in the whole system; because the upholding quality of the earth under an inside footing, for instance, was not the same as that under an outside pier. The practice was to lay out a footing under each pier exactly proportioned to the weight to be carried, and to increase, on the proper sides, footings, the earth under which should not be subjected to the same lateral pressure as that under others, and the effect of the pressure of the prevailing winds on foundations was always taken into consideration.

He had made no reference in the Paper to "the best system of wind-bracing," alluded to by Mr. Morison, for the reason that the outside walls of steel skeleton buildings were made precisely in the manner described. He believed that the steel frames of the first two tall buildings having vertical trusses for wind-bracing

Mr. Shankland.

were designed by himself. That was when the art was newer and when he had less experience than now. The system of diagonal rods in a vertical plane was direct and secure; but there were other methods of meeting wind-pressure as simple and as sure, which did not defeat the object for which tall buildings were erected, viz., the use of floor space for suites of rooms. It would be as reasonable to say that the foundations of all buildings should reach the living rock, as to assert that the only wind-bracing permissible in tall buildings should be diagonal rods in a vertical plane. No scientific tests had ever been made, as far as he knew, to determine the transverse rigidity of the different kinds of columns in common use, but all the practical tests in the shop and in erection proved the Gray column to be much stiffer transversely than the Z-bar or the box column with two sides latticed. With regard to the assertion that "the entire wind-stresses were resisted by webless columns in the Fisher building," the truth was that the building was filled from top to bottom with partitions parallel to the sides, made of very stiff fire-clay tile, 4 inches and 6 inches thick, often only 8 feet apart, the whole forming a bracing that would probably withstand the highest winds. As stated in the Paper, when designing the columns, he had provided for resistance to a wind-pressure of 30 lbs. per square foot over the broadest side of the building from the foundation to the cornice. The highest wind observed in Chicago had a velocity of 115 miles per hour, and its greatest velocity maintained beyond a few seconds was 84 miles per hour, corresponding with a pressure of 35 lbs. per square foot. But the supposition that even an 84-mile breeze would not cause a greater pressure than 10 lbs. per square foot over any face of the Fisher building was probably correct. With regard to the bay-windows, the building on one side had eight panels between steel exterior columns from top to bottom. Four of them were filled with bay-windows for part of the height, and the structure of those was designed to, and did, stiffen the building materially. Two of the other sides had bay-windows, one being like that described, and the other having five panels, two of which were filled with bays for part of the height.

The exterior covering of the steel columns was not a thin shell, but consisted of first, an air space; second, hard-burned hollow tile, 3 inches thick, each piece being fastened to the column with heavy copper wire; and third, the exterior face of very heavy terra-cotta, burned almost to vitrifaction. This vitrified terra-cotta was between 6 inches and 12 inches thick, and formed the best fire-proofing for steel structures yet devised.