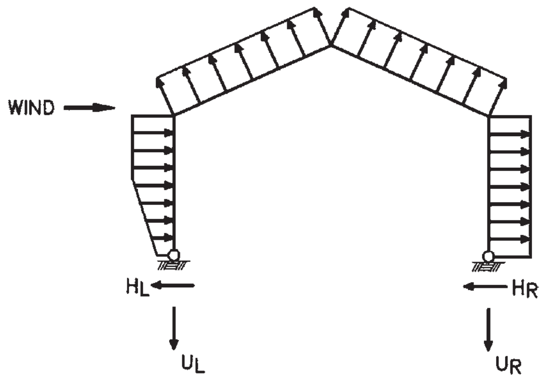

FIGURE 12.3 Uplift and horizontal column reactions caused by wind acting on gable-frame buildings.

systems, on the other hand, have extremely lightweight roofs with a total weight of only 3 to 5 psf, often not nearly heavy enough for uplift prevention.

Fortunately, column uplift can be resisted not only by the roof dead load but also by the weight of the foundation and the soil on top of it (Fig. 12.4). So, instead of making the roof heavier, it is better to increase the foundation weight, or better still, that of the overlying soil—the most economical way of doing which is to lower the footing. While this solution requires additional excavation and backfilling and thus is not quite "dirt cheap," it is often less costly than increasing the foundation footprint.

In Fig. 12.4, the uplift U is resisted by the soil weight W_1 and W_2 and the foundation weight W_3. To lift cohesive soil, such as clay, the uplift force must first overcome its shearing resistance; the plane of soil failure will generally be inclined from the vertical. In cohesionless soils such as sand the failure plane is close to the vertical line. Most engineers use a conservative approach and neglect the inclined soil segment, as well as any shear resistance of the soil.[3] If included, the angle of incline may be taken as 30° for cohesive soils and 20° for cohesionless soils per Department of the Navy Criteria.[4] Prior editions of the model building codes required a minimum factor of safety of 1.5 against wind uplift, but the modern codes are less clear on whether any factor of safety is needed at all.

In areas subjected to flooding, the "beneficial" dead load of the foundation is reduced by the water buoyancy pressure. It is not inconceivable that a flood and a hurricane will happen simultaneously, although the probability of such an occurrence should be carefully evaluated.

For deep foundations, additional uplift resistance can be mobilized by using friction piles; foundations on ledge can be anchored into the rock with drilled-in rods.

12.3.3 Foundations Designed before the Building

In conventional construction, including stick-built single-story buildings, the process of structural design normally follows a load path from the roof to the foundations. Load reactions determined for the structure at the top are applied to the lower members and, eventually, to the foundations, which are among the last items designed.

The situation is turned on its head in pre-engineered buildings. Unless the owner deals directly with the metal building manufacturer in a captive relationship, the project will probably require preparation of a complete set of contract documents. This is particularly true for projects that involve public funds. The contract documents typically include the information on both the metal building and its foundations, meaning that the foundation design must be done before the manufacturer runs a frame analysis and develops the column reactions. To add fuel to fire, some developers insist on an early "foundation contract" set of drawings to be released before the rest of the documents are ready.

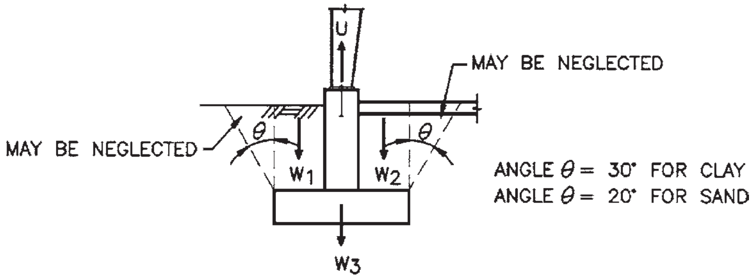

FIGURE 12.4 Development of uplift resistance.

One way or another, foundations may have to be designed before the final building reactions are obtained from the selected manufacturer. This unfortunate situation is the bane of structural engineers specifying metal building systems, who often have to design foundations based on mere estimates of the column reactions.

To soften the impact of any potential foundation redesign, some engineers choose to include a note on the contract drawings indicating that the foundation design is provided for bid purposes only, and that the actual foundation design shall be provided by the contractor, using similar details. This, of course, introduces yet another party into the project.

12.4 HOW TO ESTIMATE THE MAGNITUDE OF COLUMN REACTIONS

12.4.1 Manufacturers' Tables

The values of column reactions for a symmetrical building with standard width, roof slope, and eave height may be obtained directly from the manufacturers' design manuals. The manuals usually include the tables of column reactions as a function of the building's primary frame type, dimensions, and loading.

Typical tables for gable rigid-frame buildings with one, two, three, and four equal spans are reproduced in Appendix D. All of those assume a roof slope of 1:12, quite in agreement with our recommendations. The load tables for a tapered-beam system are included in Fig. 12.5, but note that those are based on the obsolete 1986 edition of the MBMA Manual and should be used with caution. (In general, the manufacturers' tables are necessarily based on earlier code editions.) For other types of framing, different sources must be tapped, some of which are described below in order of decreasing precision of the results they can provide.

Those who follow any approximate methods should be forewarned that final reactions supplied by the manufacturer are based on the actual sizes of tapered members and therefore will differ from the results of simplified analysis that assumes member cross sections to be constant. Indeed, even the column reactions provided by different manufacturers for identical framing are not always the same, reflecting slight variations in software, design assumptions, and the actual frame construction.

12.4.2 Specialized Software

A design firm frequently engaged in estimating reactions may consider investing in specialized software for design and analysis of metal buildings. This kind of software essentially duplicates the design process of manufacturers; it is especially appropriate if both the column reactions and the member sizes need to be known in advance. Any of several programs available on the market will