SOIL-STRUCTURE INTERACTION – A BRIEF REVIEW OF ITS IMPLICATIONS

Soil-structure interaction is a lively topic of academic and professional interest in foundation engineering. It has the potential of making designs more realistic, with its implications on economy in design. Even though its scope is increasingly being recognised, it is, however, premature to claim that it has made any profound impact in foundation engineering practice. This situation will hopefully be remedied in due course with the increasing availability of powerful softwares coming into large scale design office use.

Even though the foundation structure and the supporting soil are two different physical entities, they form a system, and in terms of the mechanics of behaviour of such systems, one component influences the behaviour of the other, the end product being the result of this mutual action, or interaction, between the two. This is the essence of soil-structure interaction. We do not propose to make anything more than a cursory survey of this subject under this Topic; those interested in delving deeper into the subject are advised first to study Kurian (2005; Ch.10) which explains the issues referred to here in greater detail, with a good number of Design Plates to assist the process of developing a right grasp of the subject matter.

15.1 Contact pressures

Contact pressures, by definition, are the reactive pressures offered by the soil on the foundation, at the interface between the foundation and the soil, against the loads transmitted to the soil through the foundation. Referring to Fig.15.1a, P₁ and P₂ are two column loads sought to be transmitted to the soil through the medium of the foundation shown. Such a medium, i.e., the foundation, has been necessary to distribute the loads on to the soil, since the soil cannot support them directly in the concentrated form as they come from above. In this process of load transmission, the foundation is subjected to a soil reaction, i.e., the contact pressure, a possible distribution of which is shown in the figure. Structurally the foundation is now subjected to bending as a result of the concentrated loads acting from above and the soil reaction acting from below. The foundation should therefore be designed for the bending produced by this system of loading and reaction. Only when the foundation is adequately designed for this condition can it fulfil the role of transmission of loads from the superstructure on to the soil in the aforesaid manner.

Even though the loads and the soil reactions together (Fig.15.1a) should satisfy the requirements of static equilibrium, the distribution of the latter can theoretically assume any form consistent with the above condition. (The requirements of static equilibrium in this case are: $\Sigma V = 0$ and $\Sigma M = 0$. The first condition requires that the total contact pressure – represented by the area of the contact pressure diagram – equals the sum
of the applied loads. The second condition is satisfied when the resultant of the applied loads and that of the contact pressure are **collinear**, so that there is **no resultant moment**. However, its actual distribution in a specific problem is the result of the **foundation-soil interaction**, which can only be determined by an **interaction analysis** involving the elastic properties of both the foundation and the soil. Thus these **contact pressures**, in the first place, are **statically indeterminate**. But whatever be its distribution, the **average** value of the **contact pressure** is statically determinate, and is simply obtained by dividing the resultant column load by the area of contact of the foundation with the soil, if the resultant load is concentric.

The **contact pressures** shown in **Fig.15.1a** are the **reactive** pressures with which the soil acts on the foundation, and thereby forms part of the ‘free body diagram’ of the forces acting on the foundation for which it is to be designed. The more **primary** question, however, is, what is the **action** of the foundation on the soil. The above means that, as far as the soil at the interface is concerned, the foundation exerts a pressure on the soil, which is **equal in magnitude** to, but **opposite in direction** of, the **contact pressure**, as shown in **Fig.15.1b**. In physical terms it means that a foundation designed for the above contact pressures will **distribute** the superimposed loads on to the soil in this manner. In other words, this is the manner in which the superimposed load on the foundation is **felt** by the soil, so to say, as it is transmitted through the medium of the foundation.

Since **contact pressures** are the result of the elastic response of the soil to the applied load, the best and the most rigorous approach to the determination of the magnitude and distribution of contact pressures is the one based on the ‘theory of elasticity’. Notwithstanding the same, there are simplified theories, such as the ‘theory of subgrade reaction’ which have been developed to deal with this problem at a more practical level.

**15.2 Perfectly flexible and perfectly rigid footings**

A **perfectly flexible footing** is one that cannot withstand any **bending moment** or **shear force**. Since such a footing has little or no stiffness, it can undergo any amount of **deflection**. Its flexural rigidity $EI$ is zero, which means that it has a thickness **approaching** zero, even if it has a positive value for $E$. Physically, a **very thin** membrane will represent the case of perfect flexibility. On the other hand, a **perfectly rigid footing** is the one that can withstand enormous bending moment or shear force with hardly perceptible deflections. Under load, such a footing settles bodily or undergoes only **rigid body movements**. Its flexural rigidity **approaches** infinity. Physically, a **very thick** block represents the case of perfect rigidity.

**15.3 Contact pressures under perfectly flexible footings**
Because of the attribute of perfect flexibility, we can actually obtain the results from simple physical reasoning without having to invoke any theory. Further, the result pertaining to this case is the same, whatever the soil, or stage of loading.

The physical requirement on perfectly flexible footing is that, since it cannot withstand any bending moment or shear force, the loading on it must invite a reaction distribution such that together they will not induce any moment or shear in the footing. This is possible only when the reaction distribution is identical with the distribution of loading itself, as shown in Fig.15.2. This means that a perfectly flexible footing cannot distribute a concentrated load, as the reaction, if any, would be concentrated (assuming for the purpose of discussion that the footing can withstand concentrated loads and reactions) and not distributed. It will be obvious while discussing rigid footings that a footing can distribute a concentrated load on to the soil only to the extent of its rigidity.

15.4 Contact pressures under perfectly rigid footings

Consider a perfectly rigid footing carrying a central concentrated load (actually a portion of a continuous footing carrying a central line load, so that the problem is two-dimensional). The results are the same whether it is a concentric (central) concentrated load or any system of loading symmetric about the centre line (Fig.15.3) so that the resultant is a concentric load.

As for soil, it should be a perfectly isotropic, elastic half-space. (We have already noted that, between stiff clay and dense dry sand, a stiff clay satisfies the definition of such an elastic medium more closely because of the seemingly good continuity of the medium due to cohesion, than dense sand which is a particulate system whose continuity is purely the result of the mechanical contact between the particles (Sec.9.4.2.))

Analysis by the theory of elasticity has shown that the contact pressure distribution below a rigid footing on an elastic soil is as shown in Fig.15.3, with minimum intensity at the centre and maximum at the edges. The latter theoretically approaches infinity, but closes in at the value of the ultimate capacity of the soil. Such a reaction distribution induces the maximum bending moment at the centre of the footing, because of its maximum dissimilarity with the loading distribution; that is to say the footing has invited a reaction distribution consistent with its very high moment capacity.

The above result physically follows if we consider the elastic settlement diagram under a perfectly flexible footing shown in Fig.15.2, which is trough shaped. Now, what we know of the settlement under a perfectly rigid footing is that, even under a central concentrated load, it should be uniform as shown in Fig.15.3. Such a uniform settlement of a perfectly flexible footing will result, if it is loaded with increasing intensity of load towards the edges, as shown in Fig.15.4. This will then invite an
identical reaction distribution as seen in the figure, which is of the same nature as the contact pressure distribution under a perfectly rigid footing.

As regards analysis, it may be noted that the above result follows when a uniform settlement is input in the analysis, rather than analysing a footing of considerable thickness.

We shall now examine the change in contact pressure diagrams when the same load applies on footings of increasingly reduced thicknesses – from an infinitely rigid footing to a perfectly thin membrane. The result shown in Fig.15.5 reveals that the contact pressure distribution become increasingly identical with the applied load. Further, whatever the distribution, the average pressure is the same, since the load is the same. Note that in all these cases, the soil response is elastic since the load which we have applied is of the order of one-third of the load that would cause a failure of the soil in bearing capacity.

15.5 Contact pressures by the theory of subgrade reaction

The theory of elasticity, which treats both the foundation and the soil as elastic continua, (continuous elastic materials), provides the most rigorous approach to the solution of interaction problems. However, the same is extremely complex – more so if one attempts closed-form theoretical solutions – for the solution of practical problems met with in foundation engineering practice. This provided the necessary impetus for developing simplified theories to meet the requirements of ordinary problems of analysis and design. The theory of subgrade reaction constitutes such a simplified approach for the determination of contact pressures below foundations interacting with soil. But the reader may be warned early that even this theory is sufficiently complex for all but simple problems of analysis and design.

In the term ‘subgrade reaction’, ‘subgrade’ is the soil beneath the foundation and ‘reaction’ means the soil reaction on the foundation. Hence subgrade reaction simply means contact pressure.

The ‘theory of subgrade reaction’ was originally due to E. Winkler and was introduced in 1867. The whole theory is built on the simplifying assumption that contact pressures are directly proportional to the deflection of the elastic system. Thus, if we denote contact pressure by \( p \), and the elastic deflection of the system by \( y \), it simply means:

\[
p \propto y = k \cdot y \quad (15.1)
\]

Where \( k \) is the constant of proportionality between \( p \) and \( y \).

The above relationship shows that if \( y \) of an elastic system is known, \( p \) follows (Fig.15.6) provide the value of the constant of proportionality \( k \) is known. (Actually,
since $p$ is a constant times $y$, the $y$ diagram itself represents $p$, but to a different scale.) But then, the elastic deflection $y$ of an elastic system is known only if $p$, the contact pressure also is known, as $y$ depends on both the loading and reaction. Thus a mutually dependent or interactive situation develops which renders the determination of contact pressures no more a simple direct exercise for anything but some simple ‘rigid’ problems.

$k$ is in the nature of an elastic constant for the soil, and is termed the “coefficient of subgrade reaction”. Since $k = p/y$, the unit of $k$ is $[F/L^3]$. $k$ is also called the ‘modulus of subgrade reaction’, ‘subgrade modulus’ or simply ‘foundation modulus’. Thus in place of two elastic constants $E$ and $\gamma$ in theory of elasticity, we have the single elastic constant $k$ in the theory of subgrade reaction. But notwithstanding the fact that both $E$ and $k$ are elastic constants, there is a basic difference between the two which needs to be clearly recognised. Thus, while $E$ is purely a material constant, $k$ depends not only on the material (i.e., the soil) but also upon the dimensions of the foundation in contact with the soil. The latter makes for an extra dimension of complexity in the use of the theory of subgrade reaction for foundations interacting with soils.

The theory assumes that the $p-y$ relationship is linear (Fig.15.7) so that $k$, which is the constant of proportionality, is the same for all values of $p$. It also assumes that $k$ has the same value for all points of contact between the foundation and the soil. These assumptions (which are no longer necessary in the numerical treatment of a problem, especially by the Finite Element Method), are collectively referred to as ‘Winkler’s hypothesis’ and the model of the soil that satisfies the Winkler’s hypothesis is called the ‘Winkler model’ for the soil.

The physical picture of the soil emerging from the Winkler model is a medium consisting of an infinite number of linear, elastic, identical, but independent springs. In examining such a model, we see that the perfect elasticity of the spring ensures the linearity between $p$ and $y$, and the fact that the springs are identical indicates that the springs have the same spring constant.

However, the assumption with the most far-reaching significance is the one pertaining to the independence of the springs. A system of independent springs means that each spring can deflect independent of the adjacent springs, due to the load acting directly on it alone (Fig.15.8a). This implies that the soil has been assumed as a discrete or elastically discontinuous medium. While this may be more true of a medium like sand, a material like stiff clay behaves like an elastic continuum, where shear interaction takes place in the soil in the vertical direction, which makes the final deflected profile more continuous (Fig.15.8b) and not abrupt as depicted by Fig.15.8a. It is obvious that a system of interconnected springs (see Fig.15.9) can represent this situation, which independent springs cannot.

It should, however, be clearly appreciated that the above limitation with regard to continuity applies only to soil, and that there is no such assumption involved on the
part of the structure (i.e., the foundation) which has been accepted as an elastic continuum in its own right (represented by its elastic constants $E$ and $\nu$), because of which the deflected profile of a beam, even when the loads and reactions are concentrated, is smooth (Fig.15.9a). Therefore when such a beam is loaded on soil (Fig.15.9b) the deflected profile will assume a smooth shape even under the assumption of independent springs.

Notwithstanding the above limitation, it is certain that, between the structure and the soil, the latter is less continuous or more discrete than the former, and to this extent we are certainly on safer ground with regard to our assumption. On the other hand, if we were to account for the continuity of the soil medium also, much of the simplicity which the theory of subgrade reaction allows for would have been lost, and one would much rather revert to the theory of elasticity approach which treats the soil also as a continuous elastic medium. However, between the limits of these two approaches, attempts have been made at successive stages to suggest modifications on the Winkler model, many of which offer only little refinement of the result when compared to the mathematical involvement of the model concerned.

Kurian (2005: Sec.10.4) describes the influence of the plan dimensions $B$ (width) and $L$ (length) and the depth $D_f$ of the foundation on the magnitude of the subgrade modulus, $k$. The final results are:

$$k (B,D_f) = k_{0.3} \times \frac{0.3}{B}, \text{ for cohesive soil,}$$

(L:B to $\infty$)  \hspace{1cm} (15.2)

and

$$k (B,D_f) = k_{0.3} \times \left(\frac{B + 0.3}{2B}\right)^2 \times (1 + 2 \frac{D_f}{B}), \text{ for cohesionless soil}$$

(L:B to $\infty$)  \hspace{1cm} (15.3)

In the above $k_{0.3}$ is the ‘unit value’ of $k$ determined by test using a square plate of size 0.3 x 0.3 m on the surface ($D_f = 0$).

Winkler’s theory also covers horizontal subgrade reaction on vertical structures such as retaining walls. It is, however, not proposed to go into this topic in this Section, the same having been amply covered by Kurian (2005: Ch.10).

15.6 Experimental determination of the subgrade modulus

If we want to determine the surface value of $k_{0.3}$, we have to conduct a plate bearing test using a rigid square plate of size 0.3 x 0.3 m on the surface, and draw the load-settlement diagram (Fig.15.10). The load on the plate need not exceed one-third the load causing the soil to fail in bearing. If the soil behaviour were linear as assumed in the Winkler’s hypothesis, the slope of the load-settlement diagram would have given the value of $k$. But since the soil behaviour is far from linear, even within the elastic range, one is beset with the problem of identifying a unique value for $k$, since corresponding to each point on the load-settlement diagram, one gets a different value.
for \( k \). One is therefore constrained to \textit{arbitrarily define} \( k \), either as an ‘initial tangent modulus’ (slope of the load-settlement diagram at the origin) or as a ‘tangent modulus’ (i.e., slope of the tangent to the curve) or as a ‘secant modulus’ (i.e., slope of the secant to the curve–\textit{secant} to a curve \textit{geometrically} means the line joining a point on the curve to the origin) at a point on the curve, corresponding to any specified value of either a load or a settlement. Between these possibilities, however, since a se\textit{c}ant at any chosen point would be more definite than the tangent at the same point for manual drawing, \( k \) is normally taken as the \textit{secant modulus} corresponding to a settlement of 1.25 mm, as shown in Fig. 15.10. It may be noted that taking \( k \) in this manner amounts to \textit{replacing} the actual curve by the idealised straight line, which is the secant, and then taking the \textit{slope} of this straight line.

15.7 Contact pressures under rigid foundations by the theory of subgrade reaction

The theory of subgrade reaction approach to contact pressure determination under \textit{rigid foundations} yields very simple results. This is on account of the fact that the settlement of the footing is \textit{uniform}, if the resultant of the loading is \textit{concentric} (Fig. 15.11a) or \textit{uniformly varying} (the net effect of uniform settlement plus tilt) if the resultant load is \textit{eccentric} (Fig. 15.11b).

And since \textit{contact pressures} are directly proportional to \textit{settlements}, the \textit{contact pressure} diagrams have the same shape as the \textit{settlement} diagrams, the magnitude of contact pressure being \( k \) times the settlements. (As stated before, this means, the \( y \)-diagram itself can represent the \( p \) diagram, but to a different scale.) What is, however, more interesting is the fact that we can obtain these \textit{contact pressures} directly from simple statics, i.e., without having to know the settlement and without invoking the theory of subgrade reaction. Accordingly \( p \) (uniform – Fig. 15.11a) is obtained by setting \( P = p \cdot A \), while for obtaining \( p_a \) and \( p_b \) (Fig. 15.11b), we need invoke only two conditions of static equilibrium, viz., \( \Sigma V = 0 \) and \( \Sigma M = 0 \), the latter ensuring \textit{collinearity} of the resultants of the applied load and soil reaction.

The important result that emerges is that \textit{contact pressures} under \textit{rigid} foundations are \textit{statically determinate} by the theory of subgrade reaction, which we know to be highly indeterminate by the theory of elasticity approach which gives a highly non-uniform contact pressure distribution under a central load (Fig. 15.5). But when it comes to ordinary footings (footings of finite rigidity) the theory of subgrade reaction gives rise to a contact pressure distribution which is \textit{statically indeterminate} as a result of the \textit{soil-footing interaction}. However, qualitatively, if we consider such a footing subjected to a central load, it is obvious that the footing will deflect with greater settlement at the centre, as in Fig. 15.12, with the contact pressure distribution also necessarily taking a similar shape. This is much different from the \textit{uniform} pressure distribution we assume in the \textit{conventional} design of the footing irrespective of the rigidity of the foundation.
15.8 Analysis of foundations of finite rigidity by the theory of subgrade reaction

Under this case, since an additional factor, viz., the *flexibility* of the foundation enters into the analysis, this case is rather involved, being *interactive* and hence *indeterminate*, unlike the *rigid* case which was simple enough for analysis under the theory of subgrade reaction.

Kurian (2005: Sec.10.7) starts with the case of an *infinite* beam supported on a *Winkler foundation* and carrying a central concentrated load. It proceeds with the analysis and gives the solutions of $y$ (deflection/deformation), $p$ (contact pressure), $\theta$ (slope), $Q$ (shear force) and $M$ (bending moment) with diagrams showing their variations. It is interesting to note that all these effects are in the nature of *fast decaying periodic functions*.

The above case actually applies to a *railway track* (Fig.15.13). In fact this was the *first* practical problem which was analysed by the theory of ‘beams on elastic foundation’ (by Zimmermann in 1888). In treating railroad as a beam on elastic foundation, the rail is obviously the beam, with the sleepers, ballast and the subgrade together constituting the elastic foundation. And track stresses are just the (static) bending stresses induced in the rail by the wheel load.

Just as we have *beams on elastic foundation*, we also have *plates on elastic foundation* as well as *shells on elastic foundation*. If a rail is a beam on elastic foundation, a typical example of a plate on elastic foundation would be a rigid pavement slab and Westergaard’s classical analysis (1926) of wheel load stresses in a rigid pavement slab treats it as a plate on elastic foundation. A flat slab raft, as also an individual footing, are examples of plates on elastic foundation, and the *flexible design* of these foundations must treat them to be so. The most complex problem of this type is the shell on elastic foundation, the obvious example of which is the shell foundation (Kurian, 2006).

Kurian (2005: Sec.10.8) describes a number of advanced models with varying degrees of refinement all of which provide for some amount of *shear interaction* by suitably incorporating a structural element in the fabric. Sec.10.8.1 also presents the author’s attempt to physically interconnect the independent Winkler springs by welding the joints (‘welded spring model’ – Fig.15.14). Sec.10.8.2 also presents his efforts to *non-linearise* the original *Winkler model*, by *fitting* the load-settlement data from the load test on the plate *hyperbolically* and using the *secant modulus* obtained from the fitted curve corresponding to each load stage used in the analysis, and plotting the *collated* results.

15.9 Soil-structural interaction: some additional observations
The interaction between the foundation and the soil stems from the basic notion that the response of an elastic system, like the foundation-soil system, comprising two elastic components, viz., the structural foundation and the soil, under load, depends on the mutual, or interacting, elastic effects of the components of the system. (The ‘elastic system’ in general can have two or more components.) We have seen this effect in relation to the theory of subgrade reaction where we noted that \( y \) is dependent on \( p \) also, as much as \( p \) is dependent on \( y \), or in other words, we cannot determine \( y \) independent of \( p \). The effects are thus found to be not independent, but mutually dependent, or interdependent, or interactive. And herein lies the concept of true interaction in the problem.

All interaction problems are elastic, (since we discuss the interaction in the elastic stage of response) and statically indeterminate. In the two approaches to the solution of interaction problems that we mentioned, viz., the theory of elasticity (i.e., ‘elastic half space’) approach, and the theory of subgrade reaction approach, a case like a rigid foundation on an elastic soil, subjected to a central vertical load is statically indeterminate, or interactive under theory of elasticity, but statically determinate or non-interactive under the theory of subgrade reaction. In effect, it depends entirely on the manner in which a problem is approached. The same can be analysed as a rigid non-interacting system as done in conventional design, or as an elastic system as would be done in a flexible design, which must necessarily take into account the elastic interaction between the foundation and the soil.

Interaction problems, whether static or dynamic, are essentially elastic; therefore ultimate problems in soil mechanics like “bearing capacity” and “earth pressure” are, strictly speaking, non-interacting, notwithstanding the foundation-soil contact. This implies that all problems involving a structure in contact with soil need not be interactive, or in other words, mere physical soil-structure contact need not necessarily connote interaction. Thus, when we discuss bearing capacity, we are actually discussing the ultimate capacity of the soil, to bring about which we need a medium to load the soil with, which is our foundation; and in discussing bearing capacity, it is implied that the foundation is infinitely stronger than the soil, thereby ensuring that, what fails is the soil, the foundation remaining intact throughout the loading. In a similar way, earth pressure problems are ultimate problems, and hence non-interacting, as once the deformations necessary to mobilise the above limiting pressures are reached, further deformations will not influence the pressures any more. Also, unlike in the case of elastic interaction problems, one does not discuss the magnitude of deformations in respect of these ultimate problems, unlike elastic problems where strain constitutes a vital topic of discussion along with stress. It may be instructive to note in this context that the water tank problem is non-interacting, since the pressure exerted by water (\( \Theta = 0 \)) remains the same, whatever be the deformation of the tank wall.

Kurian (2005: Ch.10) presents a large number of Design Plates analysing a variety of shallow foundations, by the finite element method, using ‘elastic foundation’ (theory
of subgrade reaction) and ‘continuum’ (theory of elasticity) approaches, which show how the results differ from each other and from the conventional approach without invoking interaction. Attentive readers are well advised to go through them carefully to appreciate quantitatively the extent of differences in the results.

Since advanced softwares, making use of finite element analysis, are commercially available today, which can solve problems by the rigorous theory of elasticity approach, use of simplified models, such as even the Winkler model, is slowly receding into the background, ushering in, as it were, a new era of unprecedented refinement in analysis, eventually paving the way for the more realistic flexible design of foundation systems.

15.10 Influence of the rigidity of superstructure on the differential settlement of foundations

In this closing section we shall take a look at the influence of the rigidity of the superstructure on the differential settlement of foundations.

Differential settlement is a matter of serious concern in the case of framed structures in reinforced concrete as even small amounts of differential settlement can cause a substantial amount of redistribution of loads in column elements, and moments and shears in beam elements. Such redistribution, however, has a wholesome influence on differential settlements themselves (interaction!), and it is this aspect of interaction that we are going to examine in the following.

For the purpose of our discussion, we shall consider a simple statically indeterminate system, viz., a continuous beam, continuous over three supports, that is to say, two equal spans (Fig.15.15a), and carrying a uniformly distributed load. We are not bringing any soil into the picture right now, and are considering only the superstructural system consisting of the continuous beam and its supports. When these supports are at the same level, we know from Strength of Materials that the support reactions are as shown in Fig.15.15a, Case I. If we now allow the central support to sink by a small amount, the load (or reaction) on the sinking support will reduce, and a corresponding increment of load will be felt on the two end supports, to satisfy equilibrium. At a very small value of sinking of the middle support, the support loads can reach a stage shown in Fig.15.15a, Case II. The sinking of the middle support will cause an even more significant redistribution of moments, but we are not bringing this aspect into consideration for the purpose of our present discussion.

Let us now assume, for the purpose of discussion, that the supports mentioned above are columns, and that these are founded on soil, using footings of identical size (Fig.15.16). Reverting to Fig.15.15a, if we now assume that there is an undiscovered pocket of compressible soil below one of the columns, say the middle one, which incidentally carries the higher load, it is inevitable that this column should settle more than the side columns on account of both the weaker soil under it and the heavier load
on it. This tendency to higher settlement will, however, be automatically prevented, because such a higher settlement will entail a reduction of load on this column. But this reduction will now be felt as increments of loads on the side columns. The result is that the central column will settle by an amount less than calculated on the basis of the original load, while the side columns will try to settle more than the originally calculated value. Since the tendency for higher settlement of the side columns is now prevented in the same manner as the central column earlier, it results in a transfer of part of the load on to the central column. Thus a continuous process of transfer of loads between supports sets in (conceptually), till equilibrium is struck at the end. These stages are depicted in Fig.15.15b, which shows the original position, an intermediate position and the final equilibrium position. The interesting result of this that can be seen from the figure is that the final differential settlement between the central and side columns is much less than the value originally calculated.

If we examine the above problem closely, it can be realised that the load transfer and the eventual reduction in differential settlement, has been possible because of the continuity (statically indeterminate) of the superstructure. This becomes obvious if we now consider an equivalent statically determinate system, which can be produced by simply cutting the continuous beam over the central support (Fig.15.15c), which converts it as a system of two simply supported beams. Because of the loss or continuity of this system (due to which it is statically determinate,) no reduction of column loads will be possible, whatever be the settlements of the columns, and consequently there will be no modification of the picture of differential settlements in this system. Thus it is noted from the above that the rigidity of the superstructure has a restraining and redeeming influence on differential settlement, which is a very useful result indeed.

Summarising, we may state that, on the one extreme, if the superstructure is perfectly flexible, and therefore statically determinate, settlements and differential settlements will take place unhindered, while on the other extreme, if the superstructure is perfectly rigid, settlements can occur, but differential settlements cannot (in a symmetric case), since the superstructure can partake only of rigid body movements. And in the case of intermediate rigidity, differential settlement causes a redistribution of forces in the superstructure, while the redistribution itself causes a restraint on differential settlement, and a picture of interaction emerges, the net effect of which is that, the differential settlements are very much evened out, thereby relieving the structure of much of the harmful consequences of unrestrained settlements.