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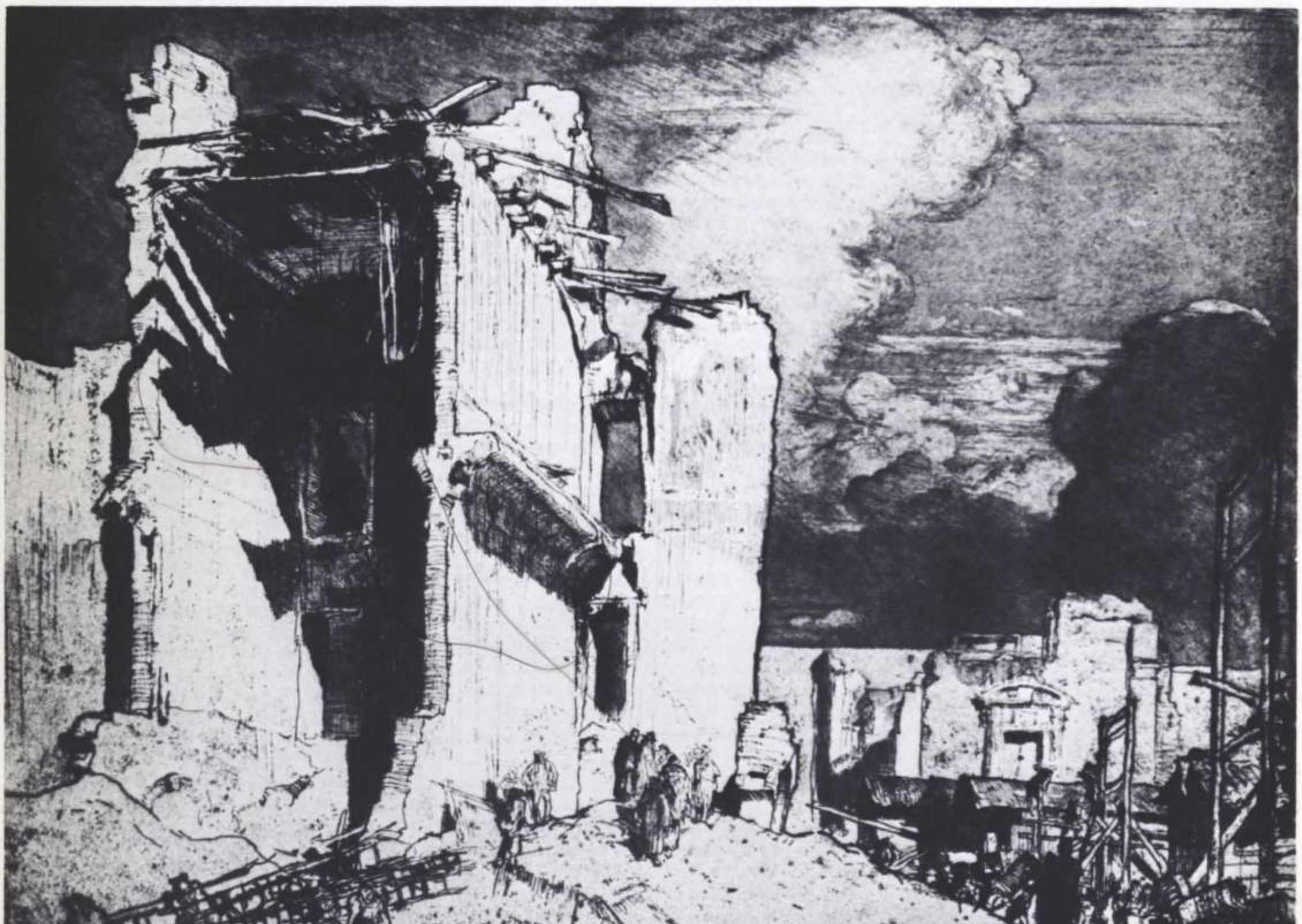

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Front cover: Etching by Frank Brangwyn showing the rebuilding, after an earthquake, of the church Il Spirito Santo, Messina. (Photographed by Tom Evans with the permission of the owner, Roger Rigby).

Back cover: Detail of Upton Priory church roof. (Photo: John Tyrrell).

The built environment

Ove Arup

This paper was given as the Building Services Engineering Society inaugural speech, at the Institution of Civil Engineers on 26 October 1972.

When I was asked to speak at this inaugural meeting of the BSES I little knew what I would be letting myself in for. I was told that the Society was formed by 10 sponsoring bodies and seven affiliated bodies to advance and disseminate knowledge in the field of building services engineering and to foster co-operation between all those involved with the total 'built environment'. But when I found that neither the RIBA, the Institution of Structural Engineers, the Institution of Heating and Ventilating Engineers, the Institute of Builders, nor the Royal Institute of Chartered Surveyors were to be found among the sponsors, I was puzzled. It was explained to me that the RIBA was on the original Organizing Committee of the Society, but was, in the event, unable to become a sponsoring body but that it was hoped that they would come in before long.

This explanation still left me puzzled, and I said I would have to investigate this matter further and discuss the result of my investigation in my speech. I confirmed this in a letter to Garth Watson which I will read to you:

Following our telephone conversation today I think I ought to put on paper the conclusions we reached, so that there is no misunderstanding.

I said that I could only speak at the Inaugural Meeting of the BSES if I could voice any conclusions I might reach after thinking about the whole matter of the Building Services, and discussing it with members of the various bodies connected with the building industry as a whole.

Whilst I concede the need for close integration of the work of all those concerned with the production of buildings, I have doubts about the value of a society which does not embrace the key professions represented by the RIBA, the Institution of Structural Engineers and the IHVE. As you say, discussions can do no harm unless they are a substitute for action, but what seems to be needed is an institution which can map out a better training for those concerned with building services of all kinds. It could most naturally be based on the IHVE, and it might be wrong to deny them a Charter, provided their standards are raised and provided only those taking the new degrees can call themselves Chartered Engineers.

I understand that you would want me to speak at the Inaugural Meeting even if I should reach some such conclusion. But if you are doubtful about this I am very willing to withdraw.

I received a reply to this letter saying that the Chairman and Garth Watson were in no doubt whatever that they wanted me to speak at this meeting as the Society was a forum for discussion and was not in itself taking any particular point of view on what were certainly controversial matters. Which I must say is a laudable attitude.

I also received a statement of 925 words on the origins of the BSES and the actions taken by the CEI in regard to the learned society and the qualifying role, ending up with an announcement that I had accepted an invitation to speak at this meeting on a subject of my own choosing *'within the theme – the built environment'.*

This last qualification is not exactly what I agreed to, but let that pass.

In the meantime I had had other letters and messages, and I had talked with various people concerned with the matter, and it was obvious that there was a large number of people who thought that the forming of this new society was not only useless but directly harmful. They regard it as a clever device by the big three, the Civil, Mechanical and Electrical Engineering Institutions, to divert attention from what was really needed and what they wanted to prevent: the granting of a Charter to the IHVE. Opinions differed about what should be done instead, but whatever it was it would be very difficult to achieve because the other fellows wouldn't play ball. The whole situation is extremely confused, to put it mildly, with institutions, charters, societies and other bodies proliferating, but never dying. Unity is extolled, apartheid practised.

I am telling you all this to enlist your sympathy for the difficult situation I find myself in.

I could, of course, confine myself to talking about the need for collaboration between all those concerned with the built environment. I seem to have done that all my working life, stretching over half a century or so, and I suppose I could do some more of it. But isn't it a bit unkind to trot out this old war-horse? After all, we all agree on that. In all my years of campaigning I have never found anybody who disagreed with it. But *talking* about it doesn't seem to have much effect. One must somehow create the conditions which will *allow* such collaboration to take place, and one must educate members of the building team to see their own contribution *not* as an end in itself, but as a part of a common endeavour to create comprehensive, total architecture. That is what we have been trying to do inside our own firm. And therefore we know how difficult it is. And yet we are particularly fortunate in being able to foster such experiments – and they have gone far beyond that experimental stage now – inside a large engineering firm able to supply the necessary engineering experience and finance. But we are, of course, all the time up against the reluctance of clients and government departments to change established rules and procedures. Especially our insistence that our quantity surveyors must be part of the design team causes uneasiness. And yet it is so obvious that accountancy cannot create anything unless it guides what is being designed and therefore what is built. The system of over-elaborate bills of quantities produced after the design is made, or worse still, before the design is made, is directly harmful in many ways, among others because it erects a barrier between the designer and the builder. One thing we ought to be able to agree on is, that the designer must know how his design can be executed, and the approximate cost of it. If, instead, priced bills of quantities are treated as secret documents which must not be shown to the designer, as happens sometimes, the whole thing becomes absurd. Designing is indicating a sensible way of building, among other things.

All this is by the way, but it reinforces my opinion that more talking is not what is needed. There are enough societies and journals where people can and do talk and write. The Joint Building Group and the Junior Liaison

Organization have more or less the same aims. And if the institutions most intimately concerned with building oppose this venture, it indicates that the most pressing need is not the forming of this society, but to bring some order into the chaotic state of separate institutions, chartered or otherwise, which have been created in a very accidental way in response to technological development and specialization, or else because groups of engineers have been dissatisfied with the conservatism of old institutions.

Now, it is obviously not very pleasant for me, having been invited to speak at the inauguration of a new society sponsored by so many worthy people, to come and tell you that the whole venture is worthless, to put it much too bluntly. It is, to say the least, an odd way to inaugurate a new society.

It would make it easier, of course, if I could also tell you what you *should* do. But I am not as clever as that. When it comes to the unravelling of the tangled network of institutions I am singularly inept, in fact. I know only a few of them, most of the letters behind names are meaningless to me. I am a bad institution man. I could, perhaps successfully, put forward excuses or rationalizations for this, but it would be a waste of time, it wouldn't alter the fact.

The Royal Charter

When I look at the list of 15 chartered engineering institutions forming the membership of the CEI I feel tempted to scrap the lot and begin afresh. Divide the whole field of engineering into sections according to the nature of the work they have to do or the knowledge they have to have, and then perhaps group neighbouring sections into a number of institutions which together would cover the whole field of engineering, united by the CEI at the top. Inside each institution you would then have different grades, Chartered Engineers, Technician Engineers and Technicians, if you like. The Chartered Engineer would have a more broadly based knowledge of mathematics, physical sciences and of all the various branch disciplines inside his particular institution, specializing in one of them, but able to represent them all at the conceptive stage of original design. And so on. New techniques or fields of operation would then originate inside one of the institutional territories and might ultimately warrant the creation of separate institutions. And some old workings might be closed down.

That is what I would be tempted to do, I said. But I am totally unequipped to do it, and in any case it can't be done, and it is very doubtful if it would be desirable to do it. For when it comes to dealing with human beings logic breaks down. To force them to do what they don't want to do is counter-productive. To destroy their traditional links with the past would be wrong too. But to build on the present haphazardly disposed foundations is a very complicated business. The creation of the CEI was, however, a very significant step in the right direction. Let us hope that it can gradually sort things out. But if it is intended to limit the number of charters to 15 for all eternity, as some people believe, it can only make sense if there is a re-shuffling of existing charters or if it is a step on the way to total elimination of charters.

All this is of little help. But let me try to establish what we, I hope, can agree on, and what we are up against.

The trouble appears to be this: The Institution of Heating and Ventilating Engineers want a Royal Charter and membership of the CEI. On the one hand they feel they deserve it. They are on the way up, their importance in the building team is growing and generally recognized, they are expanding over a wider field and want to embrace all the building services, and they are doing all they can to improve their service. On the other hand, they feel they need it, they find it difficult to attract the right kind of student unless they can dangle a Charter in front of him. If the building services engineer – or more ambitious still – the environmental design engineer – has to study another two years to join a member institution of the CEI to get Charter status, however irrelevant those studies may be to his chosen career, it will have a most disastrous effect on recruitment, to quote Mr Pullinger.

But unfortunately it is getting more and more difficult to get a Royal Charter. When the CEI was created in 1965 they were given eight years in which to raise the standard of the Chartered Engineer. By 1973 they will have to satisfy the Privy Council that the corporate members of all the chartered engineering institutions in the CEI have reached the required standard, otherwise *their* charter may be withdrawn. And they have had a look at the qualifications of the present corporate members of the IHVE to find out whether they meet the criteria for constituent membership of the CEI.

Unfortunately an *ad hoc* committee decided that less than the required 75 per cent did so, so they could not recommend the IHVE for membership of the CEI. And as a result they could not get a Royal Charter either. It seems to be the case that no engineering institution will in future be able to obtain a charter without satisfying the CEI criteria.

Whether the BSES was launched by the Civils as a sop to the wounded IHVE I don't know. It has really nothing to do with the Charter business, because the BSES is supposed to be only a talking shop, or perhaps I should say a learned society, and not a qualifying body. And it was meant to include the architects, structural engineers, heating and ventilating engineers and builders, of course.

But now these last four have withdrawn from sponsorship and any form of participation. They all feel that the IHVE should have a Charter, that it is absurd that such a vital section of the building team should, so to speak, have a lower status than the other members of the team. And they believe that the formation of the BSES is distracting attention from the much more important question of improving the status and performance of the heating and ventilating engineer.

This seems to me to be a fatal blow to the BSES. For it would, I think, make sense to have a society which embraced architects, heating and ventilating engineers, structural engineers and builders, for they are the four *main* members of the building team. But a BSES without them is nonsense.

I am not claiming any great accuracy for this rough outline of the problem, and I should not be surprised if both the contending parties were dissatisfied with my exposé. But it wouldn't do any good losing myself in all the pros and cons – the fact is, that *this fraternal dispute is a setback to the much needed mutual understanding and collaboration between the various professions engaged in building.* And the whole thing is rather silly when you consider that everybody agrees that:

First: There *is* a great need for a professional building services engineer with a wider education who, as a member of the design team alongside the architect, structural engineer, etc. can make a creative contribution to the design at the conceptual stage, before the options are frozen, and that such an engineer, by studying on a scientific basis those subjects which would be most useful to him in his work, should be able to become a Chartered Engineer, and have his professional home in a Chartered institution adapted to his needs.

Secondly: It would be a very good thing to have a forum, in the form of a learned society, where all the professions and trades who work together to shape our environment could come together and exchange views.

The difficulty in the first case is, of course, that the IHVE seems to put the cart before the horse when it demands a charter *before* the majority of its corporate members have reached the required standard – if that in fact is the case.. But they can certainly claim that there is a precedent for such a procedure – in the case of the structural engineers, for instance, and probably in the case of most of the other CEI members too. So why should the IHVE be penalized? And they *need* a charter *now* to boost morale – and on balance it would, I think, be better to let them have it even if some of them were elevated beyond their proper station – after all, it is not the presence of bad but the scarcity of good engineers which is the trouble, the bad will be found out in time, and the supply of good engineers would be stimulated. But the CEI is, on the other hand, right in upholding the calibre of Chartered Engineers, it is their job to do so, and it would be wrong to devaluate the *designation* C.Eng. in the eyes of other nations. And the alternative of elevating only some of the IHVE corporate members to the blessed state is politically unacceptable. So that's where we are stuck.

When we turn to the question of the BSES, I must admit that I cannot understand why its launching should actually *delay* the granting of a Charter to the IHVE. On the other hand there are, as mentioned, other societies and institutions who are already now engaged in such multi-disciplinary discussions, so why should the Civils, Mechanical and Electrical Engineers suddenly presume to lead in a domain in which until recently they have not shown much interest? Other bodies, especially the RIBA and the IHVE, would consider themselves more entitled to take the lead in this sphere. And, of course, if it really is only a Building Services Engineering Society, then the IHVE would have a strong claim. But that is another thing I don't understand:

Why should it have this name? I thought it was to 'foster co-operation between engineers and others in all the disciplines concerned with the design, operation and equipping of buildings' – or in another version 'with the total built environment'. So why not call it the Society for the Built Environment?

To sum up this part of my talk: The quarrel is not about what is needed, but about who should do what, and what labels to put on people. And that is really a sorry state of affairs

Motivation

It stems, of course, from the schism in the motivation of a professional man.

On the one hand he wants to do a good and useful job.

On the other hand he wants – and must – make a living. These two aims tend to conflict, and that gives rise to no end of trouble.

In any situation where many people possessing different skills have to produce an artefact – and that practically covers the whole field of human endeavour in the building field – their work must be integrated if it is to produce a whole which possesses any kind of quality. This requires unselfish collaboration, and this means collaboration aimed at producing a good job and not hampered by considerations of personal glory, status or reward. But the quest for status, profit and the rest is a fact we have to live with, and it *does* interfere with the quality of practically every job, and with the quality of the whole of our environment in fact as our pollution problems testify.

Of course the two aims *need* not conflict. Quality of environment is being produced in patches without endangering the livelihood of those taking part in its production, indeed the opposite is just as likely to result. But only if the natural acquisitiveness, greed and personal ambition of man is kept under control and the quest for quality is given first priority.

Our real problems begin, however, when we realise that it is not enough to create quality in a few favoured locations, that our survival depends on our ability to create tolerable conditions for the whole of mankind without upsetting the balance of nature. But this is by the way, although it is of course this danger ahead which ought to bring us to our senses.

Through a long evolutionary process, civilized man has to a large extent learnt to keep his natural aggressiveness under control, at least in his personal relationships. We associate quite amicably with our potential rivals for jobs or promotion in our professional institutions. In fact the common interests form a bond between us, we feel friendly even towards unknown colleagues. We become a kind of brotherhood pursuing common interests, a mutual benefit society. And then the devil crops up again, our aggressiveness is transferred to the institution, internal unity is stimulated by cultivating a feeling of superiority towards lesser breeds in other institutions. Personal ambition is camouflaged as concern for the status and public image of our profession, our officers are unashamedly pursuing a policy of extending the size, the influence and the field of operations of our particular institution. It is their duty, they feel proud to do battle for what they have no doubt is a righteous cause. We move into the sphere of politics, decisions are reached through a tug-of-war between rival lobbies, not by disinterested reason.

This is a well-known phenomenon which applies to all kinds of groups able to exact loyalty from their members, whether tribes, nations, corporations, companies, religions or political factions or what have you, and it applies in a mild and comparatively innocuous form to professional institutions. This is what lies at the root of the present impasse, and this is how the affairs of the world are generally conducted – by a struggle for power of conflicting interests – and it appears to be the only way. But we know also that it can be a dangerous way, it can even lead to the destruction of mankind. Vietnam represents the ultimate in de-personalized aggression.

One could perhaps imagine a Utopia where affairs were managed more wisely. Loyalty to a narrow circle of friends, compatriots or confrères, which is a good thing in itself, would then not be allowed to detract from our loyalty to a wider entity, that of this whole planet of ours and the life it supports. We are not living in Utopia, however. But perhaps it is not too fanciful to suggest that architects, engineers and the producers of our artefacts could forget their interprofessional rivalries and concentrate on how to improve our habitat. They all profess to do so – why not do it? It entails in some cases a widening of their horizon and a sacrifice of cherished inessentials.

Titles, letters behind your name, are these decorative features so important? Status, what does it mean? Let's get this thing in perspective. Can we not agree that it is the reality behind the façade which matters? Of course we need some labels. It's no good taking your shoes to the butcher to get them repaired – one must know or must be able to find out whom to go to. Modern society depends on advertising, and its usefulness is immensely enhanced if it is truthful. But how much of it is? In most cases a label tells you very little about what you really want to know. That so and so is a doctor, yes. But is he a good doctor? Will he kill you or cure you? Someone else is an architect. But is he a good architect? Of all those with the same qualifications some are good and some bad – or shall we say not so good. It can make an enormous difference to the job you get. And when you employ somebody or consult somebody you want to know not only his technical qualifications but what sort of person he is. Can you rely on him? Does he mean what he says, is he truthful, reliable, honest, bright, friendly, easy to get on with? Titles and labels are not much use as a guide in this respect. The Honourable So-and-so could well be dishonourable, and the engineer who is a Doctor of Science may well be useless as a designer. We all know that, so why take these letters so seriously?

I know the answer, of course. Many employers take them seriously, institutions and Government departments take them seriously, your family take them seriously, so they can really mean something, even in hard cash, at least at the beginning of a career. And patience is in short supply nowadays.

I grant all that. But then we should try to make them really mean something. Make them truthful advertising.

And another thing. What is even more important than a C.Eng. is the reputation of an engineer among those who know him personally and know his work. If I wanted some really useful information about a man, I would try to find somebody whose opinion I valued and who knew the man. But that may not be easy. Then the only reliable way is to try him out. The proof of the pudding is in the eating. Of course reputations, status, fame can also be misleading. To be praised by fools – or fooled by praise – profits no one. The only status worth bothering about is your standing among those who know you and whose opinion you value. And not least, your opinion of yourself – although even that could be the opinion of a fool!

These reflections are, of course, intended as a plea to concentrate on the essential thing: *to improve the value of our work to society, and that means without a shadow of doubt, learning to collaborate.* And improving the value of our work is not a bad way to improve our status, either. This may sound smug and banal – but nobody can deny that it is true.

To those bright boys who are supposed to hesitate before studying to become building service engineers because there is no Chartered Engineer status in sight at the end of their studies, I would say that they need have no fear that they can't get a good job at the end of it all. There is, and will be for a long time, a crying need for them.

So relax, and concentrate on your studies. For I hope that you haven't interpreted my words as a disparagement of what the title C.Eng. *should* stand for. To learn what the title demands you should learn is very important, in fact I think the standard should be raised, and that we should demand of Chartered Engineers both more basic science and more knowledge of adjoining fields. But long experience has told me that it is possible to pass an examination without deriving much benefit from it. Your ability to learn while you are working is more important. That an examination you passed as a young man or boy should mark you for life and put you in a certain category is a rather absurd over-simplification.

The architect and the engineer

The same can be said about the sharp division between the image of an architect and an engineer. The terms were coined in an earlier age, they don't fit any more, and this leads to misunderstandings. I am not suggesting that we should abandon them but that we should realise what they mean or should mean *now*.

We are in the midst of a transformation of the building industry. Arts and crafts are being replaced by science and technology – or should I say science-guided design and mechanized production. The process was in its early stages when I joined the fray, but now the rate of what we like to call progress has increased to such an extent that we must change our old ways of thinking.

Science-guided design and mechanized production – technology for short – is the domain of engineers.

It is advanced through engineering design. As the art and craft of building is being swamped by technology, the engineer muscles in on the building field. This is as it may look from an architect's point of view. He was once a master-builder. After he had ceased to be a builder himself, he was still master, he knew the art and craft of building, and he could design competently and tell the builder how the work should be done. But submerged by technology he had to learn new tricks, he was bewildered, insecure. He had to listen to advice. He was still master, but he did not master the technique of building any more. And a general who doesn't know his army, an artist who doesn't know his medium, and a designer who has to choose among unfamiliar materials and processes is in an insecure position. He cannot design with confidence. And he is in danger of losing the respect of those he commands.

But the architect wanted to remain master at all costs. For he had a sacred duty to perform. He represented the client, the user, the public. It was his responsibility to see that the building served its purpose, fitted into the neighbourhood, was a joy to behold and live or work in, did not cost more than his client could afford. All this, and more, as expectations of comfort rose, all the multiplying claims of the perfect architectural solution, including his own dreams of artistic wholeness and integrity – all this had to be achieved with the ever-increasing technical aids at his disposal.

But the engineer didn't see it this way. He could see where the architect blundered, his technical inadequacies, his squandering money on architectural or aesthetic aims which the engineer did not understand. He suspected that what the architect was doing was simply pandering to his own ego at his client's expense. And he didn't see why he, the engineer, couldn't go it alone, with the aid of the contractor, of course. He could get the foundations, the walls and the roof constructed, all sound and solid and waterproof, he could put in the required services, enclose the required number of rooms with access and exit – the lot. Why should he need a long-haired architect to tart it up and add to the expense – he knew what he liked, and if necessary he could always hire a tame architectural assistant to make a nice perspective for the client.

The client and the quantity surveyor, being concerned with value for money, were often inclined to share this view. Artistic values change, what one generation cherishes the next despises. To decide in the surge of new isms what is *'timeless'* art is difficult. Although this adjective is not infrequently bestowed by critics, it is often doubtful whether it will stick. The average client cannot be expected to share contemporary artistic sensitivity, he likes what he is used to, so quite apart from technical and economic considerations, he dislikes modern architecture. He expects the architect to provide cosy old-world cottages with all modern conveniences, access by car, etc. and available for millions of new customers. Or so one would think if one read some of his complaints.

One could not expect the architect to accept this valuation. He still believed in his mission and struggled to keep aloft the banner of Architecture with a capital A.

This, I know, is a travesty of the present complex state of the engineer-architect confrontation. Like the image of Uncle Sam and John Bull, such caricatures have, however, a long life, and I wonder whether in the depths of the engineering jungle there are not tribes who still see reality in this way. And would this, I wonder again, be at the root of the idea that a successful integration of all the building services, or even a meaningful discussion of such integration, could be achieved without the participation of the architects, or the heating and ventilating and structural engineers, for that matter? Of course one must grant that there is a great deal of truth in this caricature, otherwise there would be no problem. But I believe in the architect's mission. All the same, some, or many architects, if you like, may not be good enough, cling to outworn ideas. *But architecture is important. It is about time engineers realized that engineering is useful, necessary indeed, but not enough.*

Specialization and the environment

I suppose I will have to try to explain and support this statement. But it should not be necessary for me to go into the whole question of pollution, squandering of scarce resources, overpopulation and the rest. That mankind is in a precarious situation we all realize by now. And, as Barry Commoner so convincingly argued at the recent RIBA annual conference on 'Designing for survival', the root cause of the trouble is the massive introduction of new technologies, impelled by greed and fear.

The environment created by natural forces acting in compliance with their own laws, by fauna and flora in equilibrium or by the dwelling or tilling patterns of a pre-industrial era, all speak to us in some way – it can be awe-inspiring or sinister, squalid or pathetic, it can lift up our heart or welcome us. But the environment created by uncontrolled industrial processes, the ravishing of our countryside, the pollution, the insensitive building for profit, simply disgust us. To feel at home we must feel the impact of the human mind on our environment, not the mind of the rapist but the lover.

What could save us is also technology, but wisely guided to serve humanity. But how do we, and can we, guide technology wisely. That is the question. Technology is guided by design, and designing is decision making. These decisions are made by people. And if only these people would make the right decisions we would be home and dry.

What are the right decisions? A designer, to make the right decisions, must know:

1 What he should try to achieve, and

2 How to go about achieving it.

Aims and means, for short. The engineer is not used to worrying his head very much about the first of these problems. His task is set for him – to span a river, invent a machine to make buttons, produce an insecticide to kill certain pests. He throws himself enthusiastically into the problem and comes up with an answer. The best answer he can think of. Until recently, at least, it didn't occur to him to doubt the value of his work. In fact he saw himself as a benefactor, liberating man from drudgery and fear of want. Did he not harness the forces of nature for his benefit? Did he not increase man's power, force the earth to yield its riches?

These achievements were based on specialization. And the more the engineer specialized, the narrower was his aim, the more he shut himself off from any global view of things.

Recently I attended the Fourth Fluid Science Lecture at the Royal Institution. The speaker was Mr Braikevitch, one of the world's foremost water turbine engineers, one of those who need no introduction, but whose name I had never heard of, typically enough. He talked about the development of the water turbine. He had apparently devoted his life to the improvement of this very important tool, continuing the work of previous generations. And a very full life it was, too. According to the speaker there is more to this machine than meets the eye. Water is a fickle mistress, it has to be coaxed, but like a human being it works better when given a little freedom. As every part of a turbine is inter-connected hydraulically with its neighbour, fluid engineering has to be applied right the way through so that a harmonious whole is obtained, and the efficiency is at the maximum. The research field is therefore much wider than the lecturer was able to indicate.

Obviously, trial and error, science, and feeling for the totality, the soul of the machine, all this and more went into it. Obviously he was an artist in his domain.

The aim of all that effort was to increase the output of electricity which could be obtained from a given variation in water levels.

Who could possibly object to this aim?

I remember as a child staying with my maternal grandparents in Norway and hearing people discuss a proposal to harness the largest and most spectacular Norwegian waterfall, the Rjukan, to provide electric power. I remember the sorrow and dismay they felt at the possible loss of this awe-inspiring national monument. I was sad, too, for I would for ever be deprived of the possibility of seeing this sight. It was, I suppose, the first time I had an inkling of the ecological consequences of technology, although I didn't exactly put it that way.

A trivial matter? Perhaps. But have you ever been spell-bound by the majesty of such a display in a setting of great natural beauty? It does something to you. It teaches you humility. Have we a right to deprive mankind of such an experience forever, everywhere?

One could mention hundreds of such specialized disciplines or technologies exacting complete devotion from their acolytes, with their institutions, congresses, trade journals and their heroes who need no introduction but are unknown outside the charmed circle. And all have impeccable aims. Aims which obviously benefit mankind.

And yet, when you add it all up, there seems to be something wrong. The undoubted progress seems to be somewhat patchy. It is good in parts, like the curate's egg – but taken as a whole, the curate's was a bad egg. What went wrong? Obviously, in pursuing their aims, engineers also achieved a great number of other things. Some of them perhaps relatively harmless on a small scale, but catastrophic if large-scale interference upset the balance of nature. Like the medicine that cured the fever and killed the patient. We must understand that everything we do affects everything else, and that we must consider the consequences of our actions. Efficiency in achieving our narrow aim at the lowest cost to us or our client cannot remain our only yardstick. Systems engineering and value engineering attempt to take into account the effects of a given technological decision when assessing its merit. But merit is still equated with cost-efficiency. This is something entirely different from human welfare. Engineers have been very successful in solving the problems they are faced with. Almost too successful, for we cannot resist the temptation to show off, to do things just because we are able to do them, without considering whether we really need them. During the war we were told to ask ourselves whether our journey was really necessary in view of the need to save resources for the war effort. We have a war on now, and we would do well to ask the same question.

In other words, we must pay more attention to the first problem.

What should our designs try to achieve?

We must take a critical look at the brief, make it more comprehensive. We must look beyond the narrow object and ask ourselves: What will be the ecological consequences? What about the working conditions for those who carry out the work, including their spiritual well-being, will the work provide useful employment or cause unemployment – perhaps in other countries? What effect has it on other industries? What is the cost in scarce resources? We must ask ourselves what would happen if everybody else did what we do. Would that serve humanity? The Kantian criteria for ethical conduct.

Taking this global view is a daunting task. Engineers have a big role to play in this discussion about aims, for just as it is no good doing things which serve no useful purpose or are harmful to humanity, so it is no good aiming at things which can't be done. You cannot alter the law of gravity, for instance.

I am afraid I have spent too much time in proving the obvious: that the failure of our civilization is not a failure to increase our power, but a failure to use it wisely. We must bring technology under the control of man for the benefit of man. This has been said a thousand times, of course. Both architects and engineers see themselves as fulfilling this role. Both are right, to a certain degree, but to understand their respective roles better, we must study them in the milieu in which they cannot avoid the necessity of collaborating, the urban environment.

Architects and engineers both see themselves as designers. And although the majority of engineers and a great number of architects can hardly be called that, it is the designers I am concerned with here. For the design, as I use the word, is the key to what is built; it is the record of all the decisions which have a bearing on the shape and all other aspects of the object constructed. These decisions are unfortunately not all taken by the designer but they must be known to him and integrated into a total design.

We must distinguish between routine design, which does not require any creative thinking, and what may be labelled original, innovative conceptual or creative design. Creative design must, of course, build on previous experience and contains and employs pre-designed parts, and it may even consist almost entirely in assembling such parts to create an entity. But building is always tied to locality and to the people one builds for, and they vary from case to case. The synthesis required to create an entity, a whole which economizes in means yet fulfils the aims, is an artistic process.

Art, as the Danish author Piet Hein has stressed, is solving problems which cannot be formulated before they have been solved. The search goes on, until a solution is found, which is deemed to be satisfactory. There are always many possible solutions, *the search is for the best* – but there is no best – just more or less good. *Quality is produced if the search doesn't stop at a second-rate solution but continues until no better solution can be found.*

The artist who knows his stuff – literally – knows when it clicks. Then he knows: this is the best I can do. He has his own artistic yardstick – and if he is satisfied there is a good chance that his work will make other people happy – for he should be his own most severe critic. But this statement carries no guarantee with it, for sometimes he isn't.

This extra exertion is not dependent on monetary reward, and frequently goes without, but it is indispensable if the result is to possess any quality.

All this applies to engineering design as well as architectural design – in areas where both act as prime agents. In both cases the designer is responsible for a structural entity, and in both cases he is trying to make it function well, last well, look well and cost little, or to put it differently: make it fulfil all the requirements of the brief including the aforementioned social and ecological aims, at the least cost to the community.

An engineer who doesn't care a damn what his design looks like as long as it works and is cheap, who doesn't care for elegance, neatness, order and simplicity for its own sake, is not a good engineer. This needs to be stressed.

The distinctive features of engineering are mainly matters of content – the nature of the parts and the aims.

Engineering structures are mainly concerned with the forces of nature, overcoming difficult soil conditions, retaining earth and water, containing grain and liquids, spanning rivers, creating terra firma in deep water, moving mountains and taming rivers. All very difficult but with easily defined aims.

Architects, on the other hand, have to deal with people. Cater for them, cosset them. Would you like a little more heat, or light? Do you like your living room facing west? Or would you prefer the view on the golf course? And as the people can't reply, they have to choose for them, and get the brick-bats later.

People are fickle. They differ. They quarrel. They flock together. They want privacy. They want to drive their cars everywhere. They hate other people's cars. It is quite a difficult problem. Compared to that, the actual physical obstacles to overcome are generally trivial.

But enough of that. Besides this kind of difference there is the difference in background and education and the resulting values or criteria. What the engineer sees as a structure, the architect sees as a sculpture. Actually, of course, it is both.

In building, the entity we want to perfect is not the structure or the air-conditioning as such – although that as well – it is the sum of all these parts. The engineer only designs a part of the total. His ideal structure may occupy space which is required for other purposes, it is also part of the architectural composition and therefore subject to other criteria. The ideal air-conditioning system cannot be installed because there is not enough money or because it is deemed more desirable to enable the windows to be opened, etc. The search in this case is for a comprehensive quality which is a sum of particular qualities, each measured with its own particular yardstick, but modified to fit into a general pattern.

The success of the whole undertaking depends on the right allocation of priorities and whether the resulting entity has this quality of wholeness and obvious rightness which is the mark of a work of art.

And as this sounds a bit high-falutin I will try to show you some slides which may throw some light on what I mean, or at least will relieve the tedium.

Examples

I recently found some prewar photographs and press-cuttings in an old folder, and I will show you a few of these.

1 The first example is, I think, the first building I had anything to do with. It is a small café and shelter built just behind the river wall in Canvey Island in 1932 or '33, by Christiani and Nielsen, specialists in the design and construction of reinforced concrete structures.

I was employed by them as their chief engineer in the London office. I functioned both as architect, engineer and contractor – but was severely restricted by lack of funds and lack of architectural training. But as you can see, I had an architectural 'image' derived from the Modern Movement – a kind of mock Mendelsohn, or perhaps Tecton. The steel columns supporting the canopy of the shelter roof, introduced here to restrict the view as little as possible, definitely remind me of the Penguin Pool at the Zoo – and other Zoo jobs. Although I think these came later – so perhaps I went to the fountain-head, le Corbusier. Anyhow, the circular café with windows all round, everything supported on a concrete cylinder on piles, and its extension upwards through the café in the form of six columns supporting the roof, all this couldn't be simpler, and was certainly cheap – and, I am afraid, also rather nasty. Anyway the one time I was allowed to visit the job – my place was in the office – I was depressed by the shoddiness of the cheap standard metal windows, the concrete which had received only the normal contractor's rubbing down with cement grout and cement wash or perhaps a coat of *Stic B*, the bare columns supporting the concrete drum at the back, the cheap lino and desk, the bad detailing. But I couldn't do anything about that.

The moral of it all? That architecture on the cheap by an amateur architect employed by a contractor, and a client with no money to spend, is not a good way to achieve perfection.

2 The next slide shows a model of an ambitious scheme to build a spiral tower at the end of Clacton Pier. Visitors would enter through a central lift and then gently meander down a spiral concrete ramp, passing the shops, stalls, etc. (including peeps at what the butler saw) which were arranged on the inside. Much the same idea was later adopted by Frank Lloyd Wright for his Guggenheim Museum in New York but I am not suggesting he got the idea from me! The construction was very simple, a cone of inclined columns supported on pile groups or cylinders, and supporting the doubly cantilevered concrete ramp, which in turn contributed to the stability. It was much cheaper than making a long pier deck to house the stalls. I took the model to Mr Kingsman, the owner, who resided on the Riviera in the winter. He and his family were enthusiastic, but later saner counsel prevailed. The scheme was never built.

Moral? That bright ideas too much removed from the ordinary run of the mill hardly ever get adopted. Certainly not in prewar Britain.

3 The next is a sad story, for this monster *was* actually built. The client wanted a water tower with the tank divided into five compartments, four of equal size and one larger. As his architect was unfamiliar with water tanks or reinforced concrete, he asked my firm – J. L. Kier and Company – to give him a price for design and construction.

I came up with a scheme with four circular tanks on slender circular columns, flaring at the top to support the tank-walls. The fifth tank was formed by the space between the interconnected tanks. It was a monument beautiful to behold. But then the architect got the bright idea of using the space in between the columns for an office, adding some decorative features, rims at the top and bottom of the tanks, etc., which considerably complicated the formwork.

Moral? The intervention of an architect, even if he succeeds in pleasing the client, is not always helpful.

4 Highpoint, by Tecton, is quite a different story. Here was an architect, who knew what he wanted, and how to get it. An architect, I say – there were seven young architects, all equal, trading under this name, but one was more equal than the others. His name was Lubetkin, and from him I learnt among other things that architecture involves taking infinite care over every detail, including services, fittings, and other installations. Every tile was placed in an orderly pattern of unbroken squares, floor tiles lined up with wall tiles. Lubetkin detailed the liftcage and shaft in tubular steel and netting, light and elegant, and between us we dealt with waterproofing, insulation, surface treatment of concrete, etc., in a very dilettantish way, I am afraid. It had to be cheap, and shoddiness gradually resulted, as with all these modern Corb-inspired buildings.

I dealt with the structural design, suggesting doing away with columns and beams inside the concrete box – which pleased Lubetkin – and organizing the construction, devising a special moving platform raised by jacks from which the formwork was suspended. And I had to fight the authorities about the bye-laws and concrete regulations. So between us it was complete integration of design and construction. The heating was done by Hadens, hot water tubes being imbedded in the concrete floors, a new development at the time.

Moral? Taking pains gets results.

5 The fifth is Brynmawr Rubber Factory, designed by four young architects, the nucleus of the later ACP, or Architects Co-Partnership. It bubbles over with shells, as Jane Drew described it at that time. What I would like to point out is the way the heating and ventilation were integrated with the roof structure of the main hall. The two main ducts blowing hot air into this area were housed inside two edge beams of adjoining square domes and the point is that the whole form of construction was especially chosen to make this possible, so avoiding the usual ugly ducting. That, among other things, is what integrated design means, and I can't see how that can be achieved without the architect and the structural engineer coming into the picture beside the heating consultant.

And that is the moral.

6 Lastly I show a few slides of Sydney Opera House. Utzon believed in the architect having control of every visible detail, like Lubetkin, and he undoubtedly was an architectural genius. But the organization he built up was not capable of dealing that way with a job of this complexity and magnitude. All went well as long as he was only dealing with the structure – the architecture in this case is the structure, he used to say – but integrating the unbelievably complex installations and furnishings with the structure in his exacting way could only have been done by a much more expert integrating team. The thing had to be completed without him, but it could not be done by anybody – or by Utzon himself, for that matter – to Utzon's standards. But Utzon's brilliant spatial conception has secured him a place in the architectural firmament.

The design team

Lubetkin and Utzon are what are generally referred to as *prima donna* architects, and this is meant as an insult, the image being associated with egocentric soloists throwing tantrums. Their contribution, however, is more that of a conductor – to choose another metaphor from music, so I will use the term architect-conductors. A conductor must know the score, obviously, and he must achieve a balance of sound which is faithful to the score but adds his own artistic touch to the whole. This must be hammered out in rehearsals. The conductor who only strikes attitudes and lets the orchestra get on as best it can does not deserve the name.

What does happen when the architect-conductor is mainly a visual artist, which is generally the case, is that he will allot too high a priority to sculptural or aesthetic quality. On the other hand his critics in most cases have also the wrong priorities, for they underrate visual or spatial quality for lack of visual training or sensitivity. You can't make a deaf man appreciate music. Yet spatial music *is* important. For we must build in space and in light, we appreciate the relationships of things in space and move in space and we create our own space. So a visual artist is not a bad conductor in this case, provided he has humanity and builds for people.

Leaving it to the architect-conductor to solve all his problems with the aid of his own team of architectural assistants and the specialists he consults, and the manufacturers he guides, can therefore give excellent results, depending on the master mind. But it breaks down in the case of large, technically sophisticated jobs. He must then at least have advisers and collaborators who are constantly at hand, also at the conceptual stage, for the whole way of tackling the job may depend on their advice – he cannot any more impose a visual pattern, it is the parameters, to use Lionel Brett's phrase, which govern the conception.

But as long as the various disciplines and trades involved are represented by different firms, it is difficult to involve them all at the conceptual stage. And in any case you can only involve a few key men at that stage, otherwise the whole affair develops into a design by committee. And most important, those key men must share the conductor's view of what the aim is, and must try to achieve 'the complete integration of structure and services which will best serve that aim'.

This can as a rule be best achieved if they are independent consultants and not representa-

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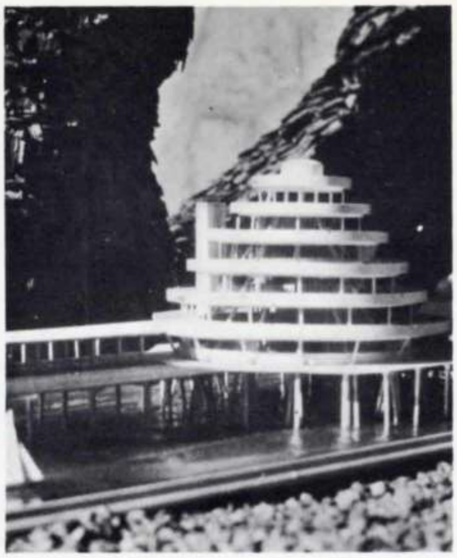
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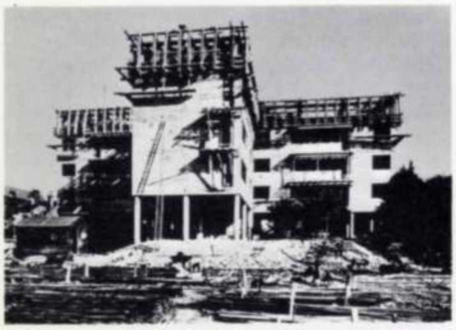
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Fig. 1
Canvey Island Café

Fig. 2
Clacton Pier model

Fig. 3
Water tower

Figs. 4, 5 & 6
Highpoint

Figs. 7 & 8
Brynmawr Rubber Factory

Fig. 9
Sydney Opera House

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tives of commercial firms. But they must not be specialists in a narrow field only – otherwise there will be too many of them. They must represent a broader section of the total technological knowledge required, able to produce – from inside their own firms, or by calling in from outside – *ad hoc* specialists as needed. They will in fact be assistant architects. Not on paper but in action. For not only should they understand the conductor's architectural ideas and approve of them, but like the architect they each represent a team covering a multiplicity of detailed knowledge and experience in a particular area of related subjects. They are also synthesizers, like the architect. It would not be a bad idea to recognize this and call them structural architects, building services architects, etc., etc., if they really are capable of fitting that role.

In the past, there have always been large lacunae in the combined knowledge of architects and consultants, which were covered by hunches and rules of thumb. That's why the fabric deteriorates, and the services don't work. This must stop, responsibility must be squarely placed in one camp or another.

The integration will then be effected from the top, so to speak, by the architect-conductor in conclave with his chief assistants and specialist co-architects, who then each will see to it that his particular team, including the *ad hoc* advisers needed, carries out the leader's agreed intentions.

Such a system can work very well, it is the parallel working which in a more or less incomplete form is generally practised.

The question has often been raised whether another profession could not fill the role of the leader. Of course, but it is not a profession but a person (with his team) which is the leader – and it depends on that person. He must then have the necessary qualifications for such leadership. He must be able to assess the priorities and effect the synthesis, be in effect an architect-conductor – in my sense.

I have not time now to discuss which technical co-architects will be required and what their role should be. There will obviously be a structural architect, for the structure and fabric of the whole building is the physical expression of the architecture. And there will equally certainly be a building services architect. As the name implies, he should cover all services, but should also understand the architectural and structural implications of their spatial requirements and the psychological and physiological effect on human beings of light and glare, of humidity, heat and radiation, noise vibration, acoustics – as well as the economic and ecological aspects of different sources of power. It is a very wide field and is largely dealt with by architectural hunches – which are very important, but can hardly deal with modern technological sophistication. One man cannot deal with all these aspects – but he should be able to call on the needed expertise as required. He would be of immense help to the building team, in fact without him expensive blunders are inevitable. And, of course, he should be able to achieve Chartered status, but his status would be assured, anyway, if he could fill the role.

And may I say again that encouraging high calibre people to fill this role is much more important than any inter-institutional rivalries. If giving them Chartered status furthers this aim, ways should be found to give it to them. If no more Charters are available, they should combine with the Structurals, who also provide a building service, or with the Mechanicals. Or some of the other institutions should combine – Structural with Civil for instance, which would be sensible – or what about Mining with Mining and Metallurgy, Marine Engineers with Naval Architects – leaving a space open, so to speak. This may be utterly naïve, but those who know the ins and outs should find a way.

I wonder, are institutions created to prevent us from doing what we want to, or to give us an excuse for *not* doing what we ought? Surely not, so let bygones be bygones, kiss and be friends and let Fred have his lollipop. And if you think I am not serious you are wrong.

There is a third chief adviser who I think is needed, especially when the design stretches into untrodden territory. It is a production engineer, or operational costing expert. But this is a chapter in itself, and highly controversial to boot. I have probably offended enough people for one evening, and I have no time, either.

There are, of course, many ways of working other than the way I have just described, in fact the possible permutations are legion. There is, for instance, the contractor's package deal, and there is the multi-disciplinary team working as we practise it in Arup Associates.

The latter is probably the best way to eliminate professional rivalries and create the right enthusiasm straight down the line, but it takes time. The quality of people plus enthusiasm is what matters, much more so than the type of organization. But the latter should be of a kind to encourage and not thwart enthusiasm.

The comprehensive view

Just one thing more – to set up an organization which is able to effect the integration of diverse elements of the environmental fabric is one thing. But its usefulness is severely restricted by existing bureaucratic boundaries.

Every time a major surgical operation disturbs established environmental patterns it sparks off side effects which may even make the proposed operation obsolete before it can fulfil its function. Where people live and work determines the transport network that establishes routes for underground services that feed the buildings with light, heat, water and telecommunications. These in turn generate new and inter-related problems and so on almost *ad infinitum*. A comprehensive view is essential. But departments keep on dealing with one aspect at a time.

The establishment of the Department of the Environment is the central Government's answer to this problem. But will this enormous conglomeration of Civil Servants and professional gentlemen be able to cope with it? It seems almost too much to expect.
We must hope that they will learn by experience, it is obviously a step in the right direction. But we are beginning to witness the emergence of pressure groups of laymen who no longer are satisfied to trust the collective wisdom of the professionals to supervise our environment. The task is to restore trust in our ability to tackle comprehensive problems comprehensively.

Our aims must be comprehensive,
Our building competent,
But priorities must be fixed by men, not by machines.

I have said both too much and too little. I have tried to place the problem in its global or overall setting and therefore courted and, I am afraid, not avoided the dangers of superficiality. But I have made it clear, I think, where I stand.

I honestly cannot see how the BSES can achieve what it set out to achieve as long as the principal institutions concerned with building oppose it. It would become a bone of contention instead of a unifying influence. And we very much need a unifying influence. I am all for a society of this kind, but the name should be something like the Society for the Built Environment. And it should not be the property of any institution. I cannot see the architects flocking to a meeting at the Civils or the engineers coming to the RIBA. The venue could be changed from time to time, meetings could be arranged also in other cities. It should, of course, be supported by all those institutions who are now sulking – no disrespect intended – and it should collaborate with or absorb the Junior Liaison Organization and perhaps other groups who have the same aim. And the governing body, or at least the body that takes the initiative, should not consist of a representative from each of the sponsoring institutions or anything of that kind. It is not their job to represent anything except common sense. To run the show we should have people who understand the problems, who are convinced of the need for collaboration and have the enthusiasm, drive and *tact* to further the aims of the society. (The last is inserted to leave me out).

And finally I suggest that the matter should be taken to the Presidents' Committee for the Urban Environment to initiate the formation of such a reformed society. This was Alex Gordon's suggestion, and it is obviously a sensible one, as you would expect. I hope the present sponsors would generously agree to that.

It remains to thank my hosts for affording me the liberty to express my views. If I should have caused offence I regret it – but I can only say what I think is right. If I am not right, then I can only ask you to forgive me. If I should have been able to convince you that my views are sound, you will be generous enough to act on them.

But I cannot help thinking of Orwell's words in the recently discovered foreword to 'Animal Farm':

'Liberty means the right to tell people what they do not want to hear'.

To which my secretary cynically added:

'Stupidity is expecting that they will listen'.

Modern construction techniques for earthquake areas

David Dowrick

Presented to the Fourth European Symposium on Earthquake Engineering, London, 5–7 September 1972.
(*Printed by kind permission of The Institution of Civil Engineers*).

Earthquake engineering will be the subject of a Technical Staff Meeting on 10 January, 1973. All Arup staff are welcome.

Introduction

Earthquakes demand the highest performance by the construction industry. The best modern antiseismic construction is characterized by:

(i) Excellence in design
(ii) Discrimination in the use of materials
(iii) Utilization of the most effective plant, and
(iv) High standards of workmanship.

In this paper, 'construction' is interpreted in its overall sense as being the process of designing and building, and the relationship between these mental and physical processes. Construction techniques are not simply those operations which are carried out on site by the contractor, but inextricably involve the design processes which dictate the nature of much of the construction work. Design decisions affect construction, not only in the initial choice of materials but also in the selection of size of components and in the detail of fabrication.

Special problems are to be found in developing countries which lack the materials, equipment or skills to utilize modern construction techniques fully. Although some attention has been given to this fundamental problem in many countries, much yet remains to be achieved.

Modern design philosophy has had a wide effect on construction techniques, on the style of construction and the manner of fabrication. Research into the strength of materials and performance of units of structures has also led to changes in site procedures. The preceding facets of modern construction techniques will now be discussed in more detail in relation to different types of buildings and bridges.

Taller buildings

Recent years have seen a considerable increase in the number and maximum height of taller buildings in zones of high seismic risk. This presupposes, among other factors, a growing confidence in our design and construction expertise. Tokyo dramatically exemplifies this change with the elimination of its ten-storey height limit in 1964, followed by the current construction of a group of buildings over 50 storeys high and a design study of a 300 m high television tower.

The construction techniques used in taller buildings are obviously dependent on the choice of materials and the rationale of desirable structural behaviour. Steel is still indisputably the safest material for tall structures, with in situ reinforced concrete following well behind in second place. Although precast concrete buildings have been built up to about 13 storeys in height in zones of high seismic risk, totally precast buildings as yet remain effectively confined to low- and medium-rise buildings. The same generalization can also be made for reinforced masonry construction.

One way of comparing the current aseismic capacity of different materials and methods of construction is through the maximum heights of buildings in each material. Table 1 gives the maximum heights (in earthquake zones) known to the author, as yet used for different materials. The table does not claim to give record heights and no competition is implied.

Table 1

Dominant structural material	Height of taller structures in zones of major seismic risk	Location
Steelwork (bending)	60 storeys*	Los Angeles
Steelwork (diagonally braced)	34 26	Los Angeles San Francisco
Steelwork (slitted concrete shear walls)	55*	Tokyo
Reinforced concrete (in situ)	43* 26 23 22 20	Caracas Los Angeles Mexico City Auckland Yugoslavia
Precast concrete	13	Yugoslavia
Reinforced masonry	15 8	California New Zealand

*Indicates under construction at the time of writing

Recently, there has been some progress in defining a coherent rationale for aseismic structural design, arising from greatly improved computer programs for theoretical studies and a valuable increment in physical testing of structural elements. Nevertheless, fully rationalized anti-seismic structural systems have not yet been developed, partly because of the continuing incomplete understanding of the significance of great stiffness and great flexibility on seismic behaviour. The greatest weaknesses in our present understanding arise mainly from our lack of data on the behaviour of total structures during strong ground motion. It is to be hoped that the post-mortems of instrumented buildings in the San Fernando earthquake of February 1971 will soon provide much-needed design information. Similarly the research programmes of the larger shaking tables should also help remedy this situation.

But what is the present rationale of aseismic construction? An attempt to answer this question is made in the following three sections.

Flexible tall buildings

Earthquake motions are so large that tall buildings are seen to behave similarly to ships and aircraft in that the cladding and partitioning must be dealt with as positively as the structure. In the construction of flexible buildings it is now acknowledged that stiff cladding and partitioning should be fully separated or articulated from the structure. In Japan, New Zealand and California this is widely practised, if not actually required by their codes for important buildings.

There are two important disadvantages of fully flexible construction with separated non structure. Firstly, there is no hidden reserve of strength supplied by the non-structure, which means that the structure must have the capacity to survive the design earthquake unaided. Secondly, such flexible structures tend to be too flexible, the lateral storey drift becoming excessive. It has allegedly been calculated that if the San Fernando earthquake had been a little closer to one particular modern steel-framed building, then that building would have suffered a storey drift of about 75 mm. Obviously no cladding separation details could accommodate such movements and presumably the building would have collapsed completely through the excessive column movements (the $P\Delta$ effect) if such movement had occurred.

Stiff tall buildings

Rather than create a very flexible structure, the opposite extreme is to construct a very stiff building which will accept all the seismic forces induced in it. This can be done either with diagonally braced steelwork or the use of precast or in situ concrete shear walls. In very stiff structures with low storey drift, the non-structure can be tied into the structure without either analytical inconvenience or danger of too much damage during earthquakes. To date, only moderately high buildings have been made of very rigid construction in major earthquake areas, mainly because of doubts on the effects of the low ductility of such structures and also because of difficulties in achieving the requisite strength.

In precast concrete construction, there are considerable practical difficulties in achieving economical and rapid jointing details of sufficient strength. Some joints of adequate strength in non-seismic load conditions have, of course, been found, but much improvement is still required in the dynamic situation. Such joints as are being used have not been proven to be properly aseismic because of the lack of real dynamic testing.

The situation in in situ shear walls is somewhat better as more shear wall buildings have survived earthquakes and more reversed loading tests have been done. A major earthquake weakness of shear walls has been the construction joints, but it has been demonstrated that proper reinforcement arrangements and simple preparation of the lower surface overcomes this problem. Another difficult aspect has been the treatment of coupling beams in coupled shear walls, notorious in earthquakes. Some encouragement has been provided by Paulay[1] who has found a strong and ductile reinforcement arrangement. Although this detail might be a bit awkward for the contractor to build, it should be little more so than lapping column bars at mid-height, a practice which is now customary in California and New Zealand.

It is felt by some engineers that orthodox concrete shear walls, if properly understood could play an important role in taller earthquake-resistant buildings, but more research into the overall behaviour of shear walls is still required to assess their strength and ductility characteristics.

Diagonally braced steelwork generally is not as stiff as heavy concrete shear wall construction, particularly if the bracing only resists tension. Therefore, for reliable prediction of the seismic behaviour of such frames, the non-structure should be fully separated from the structure. Although the diagonal bracing reduces the bending moments in the framework, it naturally increases the direct forces, with two major disadvantages. Firstly, it is not always practicable to anchor the large tensile overturning forces which occur in the bottom columns and secondly, the instability problems are not yet fully understood. There are also some current fears about early brittle failures of compression members after only a few load reversals.

Semi-flexible tall buildings

It is of course possible to construct a compromise between over-flexible steelwork and over-brittle shear walls by inserting concrete shear panels in a moment-resisting steel frame. Designers have assumed that orthodox in situ concrete walls will behave as required in this situation, and this seems likely to occur if both the geometry and detailing of the walls are suitable, but little or no research has been done on this structural arrangement. A more sophisticated variant of this has been developed and researched in Japan in the form of the slitted shear wall[2]. The concrete panels behave as rows of columns rather than as shear walls, and restrain the structure from excessive drift in moderate earthquakes and strong winds. An example of this type of construction is the 47-storey Keio Plaza Hotel in Tokyo which may be the most slender earthquake-resistant building in the world with its height/width ratio of 6:1. In very strong earthquakes, the concrete cracks and the ductility of the steel frame saves the building from collapse. This fail-safe mechanism is particularly suited to lessening the structural response from predominantly short-period ground motion. If buildings constructed this way respond to actual strong motion earthquakes in the manner predicted by the reversed loading tests, then the sophistication of the approach will be well justified.

Some quite important disadvantages are attached to the construction of flexible and semi-flexible structures. Firstly, in reinforced concrete construction, the creation of highly ductile members involves enormous congestion of reinforcement, especially in beam/column junctions (Fig. 2). This problem has been partly relieved by lapping the column reinforcement half-way up each column, but the satisfactory anchorage of beam bars in the joint zone is not yet resolved. The fixing of reinforcement and pouring of concrete in large ductile frames is becoming a construction nightmare. Perhaps engineers should stand back from this material and ask if they are using reinforced concrete properly. Is it being designed to yield too soon? Are engineers so busy forcing it to be ductile, like steel, that they are failing to profit from its intrinsic properties of stiffness and massiveness?

Flexible ductile steel frames run into some difficulty in guaranteeing that columns should be stronger than beams. Unfortunately, the yield strength of steel sections varies so widely that, even with 10 per cent nominal difference in failing strengths, the actual ratios of column strength to beam strength can be almost reversed because of variable yield strength. Unless the steel manufacturers can decrease the variation in this property of steel, no economical solution to this problem appears to exist.

The third disadvantage of flexible and semi-flexible aseismic buildings is that of the expense of separating the non-structure. It is clearly costly to provide adequate mountings and stability for both cladding and partitioning, which leaves them free to move independently of the structure. These costs may be well justified in relation to the overall cost of a given project but are nevertheless real.

Medium-rise buildings

The dominance of steelwork structure in taller buildings is not maintained in medium-rise structures with both concrete and masonry becoming competitive. Concrete and masonry are desirable for similar reasons – they are widely available, fairly easy to build with, architecturally flexible, visually variable, durable, and comparatively cheap. Unfortunately, it is their very adaptability and the apparently low demands they make on construction skill that cause them to be hazardous building materials

Fig. 1
Olive View hospital, destroyed by an earthquake in San Fernando in February 1971. (Photo: David Dowrick)

in earthquake areas. Earthquakes attack buildings which have not been specifically planned and built with suitable dynamic characteristics.

Too many medium-rise buildings have been designed to have certain rudimentary resistance to lateral forces which have been seen only as static rather than dynamic loads. This design approach may well suffice for buildings fully tailored for earthquakes, with structural and non-structural planning and detailing co-ordinated as with the best tall buildings. One of the features of the San Fernando earthquake was the great amount of non-structural damage, much of it occurring in medium-rise buildings which suffered little or no structural damage[3]. It is about time that structural engineers, architects and services engineers collaborated to organize aseismic design comprehensively.

The best examples of where this has actually been done are in buildings designed in multi-professional design offices, or in countries where the distinctions between architects and engineers are comparatively narrow. For example, in Japan, the Architectural Institute, which comprises architects and structural engineers, has published 'Guide lines for aseismic design of tall buildings'[4]. Amongst other things, this says that 'the deformation of a buildings under seismic loadings should be limited in order not to endanger the public or cause inconvenience in use of the building'. This clearly applies to *all* professions involved in building design. Although these guide lines are actually for buildings over 45 m high, similar rules should apply to lower structures.

Both concrete and masonry have suffered from misapplications, either the form of such structures being unsound or the workmanship being sub-standard. As a consequence of their poor earthquake performance, some countries have very strict limits on the use of concrete and masonry. Japan restricts reinforced masonry to three-storey structures and orthodox reinforced concrete to five storeys. There are signs, however, of an awakening in the Japanese reinforced concrete industry with design studies of buildings up to 20 storeys in height and considerable interest in precast solutions. These latter include the construction of precast silos 38 m high and seven-storey precast buildings. Suzuki has briefly reported on these as well as proposals for 15-storey precast buildings not yet built[5]. New Zealand, while progressive in in situ reinforced concrete buildings, has made small use of precast frames and has mainly limited masonry to low-rise structures. Despite strict requirements for inspection and grouting-in of reinforcement, the unreliability of workmanship remains the biggest drawback of reinforced masonry construction.

In more populous areas with large housing problems and limited resources, the attractions of industrialized building have advanced the use of precast concrete more than in affluent seismic areas. A 13-storey precast post-tensioned concrete apartment block in Banja Luka, Yugoslavia, survived the 1969 earthquake (MCS VIII, $Ms=6.1$) without damage[6]. Following the virtual destruction of Tashkent by an earthquake in 1966, 2,150,000 m^2 of fabricated housing was built in two years in buildings up to nine[7] storeys high. Industrialized buildings, with its possibilities for good quality control in the factory, concentrates many of the strength and construction problems in the joints. No fully satisfactory jointing methods have as yet been demonstrated, and much proper dynamic testing of joints is required. For these reasons, many earthquake areas which could benefit from industrialized construction have made little use of it to-date.

Low-rise buildings

Modern construction techniques have helped greatly in reducing death and destruction in recent earthquakes. Simple bracing and tying together at floor levels have been the chief contributors to this, along with improved substructure arrangements. School buildings of

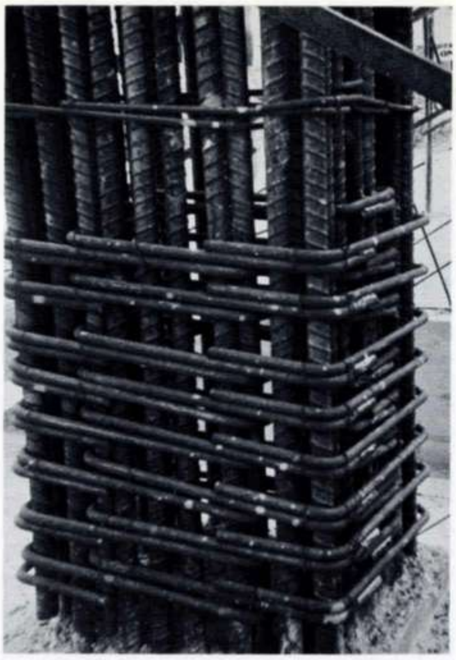

Fig. 2
Detail of column reinforcement with confinement ties required for seismic resistance. (Photo: Courtesy of Babbage and Partners, Auckland, NZ)

modern design performed very well in the 1971 San Fernando earthquake[3]. Nevertheless, a good deal of damage occurred to low-rise housing and industrial buildings in that earthquake, and also in New Zealand in the 1968 Inangahua earthquake[8], despite the common use of timber construction which generally performs well in earthquakes.

Both these earthquakes underlined one of the inherent problems of low-rise construction. It is comparatively expensive to make low buildings act as units in earthquakes because:

(i) a properly coherent substructure is very costly in relation to the total building cost;

(ii) structurally rather incompatible materials are often used in the construction of low buildings.

It is technically difficult or expensive to tie together masonry walls and timber floors or chimneys and timber walls. In low-cost housing, especially in underdeveloped countries, there is still much room for development of simple cheap aseismic construction techniques. If low-rise buildings are less effective in earthquakes than taller buildings, a major reason for this is the lack of effort put into developing their aseismic strength. The reasons for this may be two-fold. Firstly, the lack of funds made available in the past for developing the aseismic properties of low-rise buildings; with more governments now more actively pursuing low-cost housing policies this situation is gradually changing. Secondly, in the past engineers themselves have taken less interest in low-rise buildings than in higher structures. It is to be hoped that this situation also will change soon.

Bridges and elevated roadways

As can be seen with buildings, the construction techniques used for aseismic bridges are following cautiously along similar lines to those of bridges in non-earthquake areas. The major building materials of steelwork, prestressed and reinforced concrete and composite steel and concrete are used with confidence. Longer and longer span bridges are being constructed, and great lengths of elevated roadway are being built in seismic regions such as California, Italy, Japan and New Zealand.

It is interesting to contrast the uninhibited use of prestressed concrete in bridge construction with the strictly limited use of this material in buildings. This is explained by the fact that the prestressing is confined largely to the bridge decks, which generally provide a relatively small seismic design problem compared with the supporting structure. Because bridge superstructures are usually articulated from the substructure and must have great strength against vertical loading, the seismic stresses are generally modest.

Thus, the real challenge in aseismic bridge construction lies in the supporting structure and its connection to the superstructure. Recent earthquakes involving many bridges, such as those of Inangahua, New Zealand, May 1968[9], Madang, New Guinea, November 1970[10] and San Fernando, California, February 1969[11], have confirmed this situation. Bridge superstructures performed well in these earthquakes despite being designed against seismic forces which now may be considered somewhat low. For example, the San Fernando bridges were designed against equivalent horizontal static loadings of 2 per cent to 10 per cent gravity, and some superstructures survived peak ground accelerations of up to about 50 per cent. Concrete box girder decks remained intact even with large differential settlements of the foundations, and short span continuous bridges generally performed well.

Bridge approaches and abutments

Except where rock outcrops at deck level, the abutments are the most difficult part of a bridge to make earthquake resistant. Where there is any significant depth of sedimentary deposit or fill in the approaches, the abutment should be constructed to dissipate the inevitable ground movements which produce destructive forces if contained. Longitudinal battering occurs between the bridge deck and the abutments, damaging and distorting both components. Skew bridges tend to rotate about a vertical axis, becoming noticeably more skew.

Various methods of improving this situation are available. Spill-through abutments with wide gaps between the vertical members help relieve the soil pressure, but are not always desirable for other reasons, such as the difficulty of toe protection. In some multi-span bridges, it is possible to resist the deck inertia forces in the piers, designing the abutments to be free-standing. This is less likely to be feasible in the lateral direction than the longitudinal direction, and fail-safe shear keys may have to be provided.

Despite considerable improvement in compaction techniques, some settlement of approach fill should be anticipated. To prevent road vehicles running into the ends of bridges, a settlement-relieving slab should be provided on the fill at each abutment.

Bridge piers

Although not subjected to seismic soil pressures in the manner of abutments, piers are nevertheless subjected to considerable earthquake forces due to relative deck and ground movements. Four of the five bridges which collapsed in the San Fernando earthquake were of the inverted pendulum type with relatively tall piers (Fig. 3). Of all the 40 bridges which were damaged in that earthquake, some exhibited shattered pier shafts and several included pier foundation failure.

Despite the rather spectacular failures of modern bridges at San Fernando, much better performance of similar construction is possible for little extra cost. Although the ground movements and accelerations were very large, present-day knowledge and building techniques, if properly applied, could have prevented collapse and reduced damage generally. That particular earthquake is doing great service in helping the construction industry get its design criteria right. Some improvements in pier detailing which would have helped are:

(i) More column links, securely anchored;

(ii) Properly confined controlled hinge zones at top and bottom of piers; and

(iii) Piers, footings and piles properly tied together.

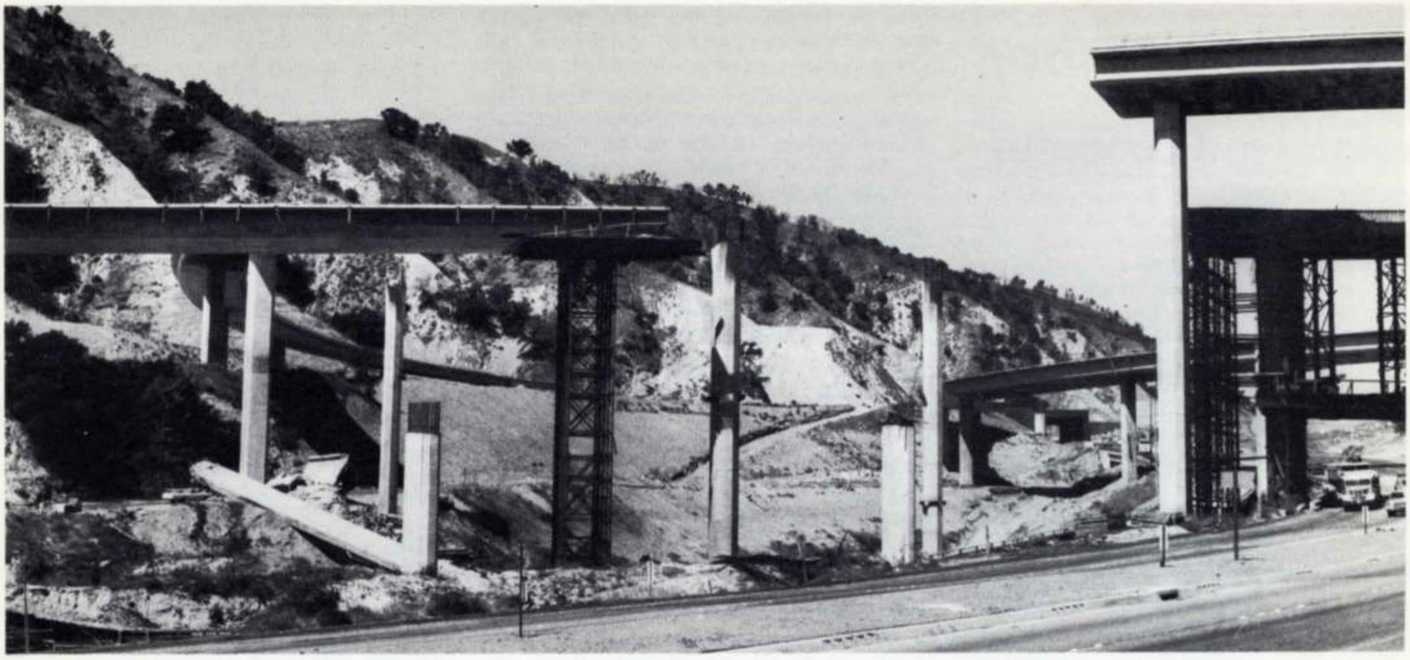

Fig. 3 above
Destruction of pendulum-type bridge in freeway interchange in the San Fernando area. (Photo: David Dowrick)

Fig. 4
Example of battering damage in a skew motorway bridge near San Fernando. (Photo: David Dowrick)

Just as with the design of buildings, not enough bridges (complete with subsoil) have been treated to dynamic analyses. How can the effects of out-of-phase relative movements of different components of a long structure, with several independent footings on different subsoil, be adequately designed with a static force approach?

Given reasonable subsoil conditions, the construction techniques for sound, earthquake-resistant piers are available. A variety of large and small diameter piling techniques exist to deal with a variety of soil conditions. Adequate provision of reinforcement in the tops of piles or cylinders is possible, which will prevent failure or hinging of piers below cap level where inspection and repair would be most difficult.

Slip-forming techniques are often used in pier shaft construction and result in efficiently constructed and elegant structures. Many fine examples have been built in seismic areas, in Italy in particular.

Bridge movement gaps and deck support connections

In terms of detailed design, these parts of bridges are probably the most difficult, in both seismic and non-seismic areas. Movement gaps and some articulation of deck from support must be provided to deal with shrinkage, creep, temperature and ground motion effects. Great advances have been made in the treatment of bearings and expansion joints, but self-cleaning, self-draining details, which allow the desired amount of free movement but no more, seem virtually impossible to design. Despite the wide variety of new materials and devices which have been tried for various components of articulation zones, the conflicting and demanding service conditions have so far prevented the development of fully satisfactory solutions.

Some of the lessons from recent earthquakes[9, 10, 11] are as follows. To prevent spans falling off at sliding supports, longer seatings are desirable. In some cases, a length of 400 mm has not been sufficient. Damage due to battering of adjacent deck portions, or deck and the abutment, can be considerable (Fig. 4) and unconfined rubber buffers may be the best provision against this. At longitudinal hinge positions, spans should be tied together to prevent the supported span falling off[12]. This is an awkward detail to design properly and is likely to be constructionally difficult in most cases.

Conclusion

In this brief review of some parts of modern earthquake-resistant construction, an attempt has been made to show the relationship between the designers' and the builders' skills, and to indicate the general level of competence in the more advanced construction industries. There is still a great need for the development of more highly aseismic construction details. Co-operation between builders and designers is desirable for achieving simplicity of detailing. Simple details not only generally behave well in earthquakes but are much more likely to be built properly.

Further efforts should be made to ensure the proper use of the major building materials. More dynamic analysis and real dynamic testing (as distinct from static load reversal tests), particularly of whole structures, will help in this regard. In situ and precast concrete and reinforced masonry are greatly in need of this type of testing. It is much to be hoped that the shaking tables can do what is intended of them. The need for adequate research and development has been particularly clearly seen by the Japanese construction industry whose work in this field could be an example for others to emulate.

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A computer analysis for slender reinforced concrete columns

John Gaughan

Introduction

The capacity of a column section can be represented by an interaction curve like *ABCD* in Fig. 2. Point *A* represents failure of the section in pure compression and *D* the failure in pure bending. Failure with a combination of compressive load and bending moment can be found by drawing a straight line such as *OB* through the origin *O* and with a slope equal to the particular ratio of end load to moment. Point *B*, where the line intersects the interaction curve, marks failure.

With eccentrically loaded columns, the moment arm depends on lateral deflections due to loading as well as on the initial eccentricity. The bending moment at a particular section will not vary linearly with compressive end load. The variation may be according to *OB′* or, for a more slender column, *OB″*. Points *B′* & *B″* mark the occasion of failure of the section. The ideal eccentrically loaded 'short' column would of course follow the line *OB*.

With very slender columns, lateral stiffness may be lost completely before actual failure of the section and in this case there is a rapid increase of lateral deflection after the critical loading has been reached. The curve *OEC* corresponds to this situation. Were it possible to control events after the critical loading at *E*, the curve *EC′* might represent the section behaviour, each point on the curve corresponding to an unstable equilibrium stage in the collapse of the column.

For other descriptions of the manner of failure, of columns, the reader is referred to Breen and Ferguson and to MacGregor and Barter[1] (Paper no. 6).

The mode of buckling for real slender columns fully built in at the base and with the top pinned and restrained only against lateral movement, is characterized by that for an ideal axially compressed straight strut with similar restraint as in Fig. 3.

The mode involves double curvature. In the case of a reinforced concrete column, the degree of cracking will depend on the curvature and the axial strain. The bending stiffness at any section will therefore depend on the degree of cracking. The bending stiffness at points of contraflexure such as *A* and *B* will be higher than at points of maximum curvature such as *C* and *D*.

Real columns are not perfectly straight and the ideal column behaviour of no lateral deflections until the critical load P_{CRIT} is reached, followed by rapid lateral deflections without any significant change of end load, will never occur. In practice lack of straightness and eccentricity of loading will cause the column to start bowing sideways as soon as end load is applied. If the imperfections of manufacture and loading are small, then, as loading is continued, a slender column will continue to bow sideways with increasing rapidity until, as the ideal critical load is approached, lateral stiffness is lost completely. A plot of column load against the lateral deflection at a characteristic point, *C*, in Fig. 3 for example, will approach tangency to the ideal critical load, as shown in Fig. 4; curve *OEC* of Fig. 2 would be a typical end load against moment curve for this case. If the column could be controlled after the peak end load, a fall in end load with further lateral deflection would be observed.

For a slender column with only small manufacturing and loading imperfections, the ideal strut critical load is seen to be an upper bound to the column capacity. Columns, like those in Fig. 3 and with constant cross-section, have an ideal critical load given by Timoshenko[8] as

$$P_{CRIT.} = \frac{\pi^2 EI}{(\cdot 699\,L)^2} \qquad 1.$$

Unfortunately as the equation only applies to columns of constant cross-section, the exact analysis of other columns presents problems. Even with a reinforced concrete column having the same overall cross-sectional dimensions everywhere, the effective cross-section allowing for cracking of the concrete will vary along the entire length; the flexural stiffness *EI* will not be constant.

The problem then is to find the capacity of reinforced concrete columns by

(a) determining the ideal straight strut critical load and regarding the load as an upper bound, or better still

(b) a detailed examination of the behaviour of the column material, allowing for the effects of manufacturing and the loading imperfections, and leading to a calculation of the ultimate strength.

The first approach has the disadvantage that at the end of the calculation there is still the problem of deciding by how much the ideal critical load should be reduced to give a safe estimate of the real column capacity. In addition, for a reinforced concrete column, the task of determining accurately the ideal critical load is of comparable difficulty to determining the ultimate capacity allowing for imperfections.

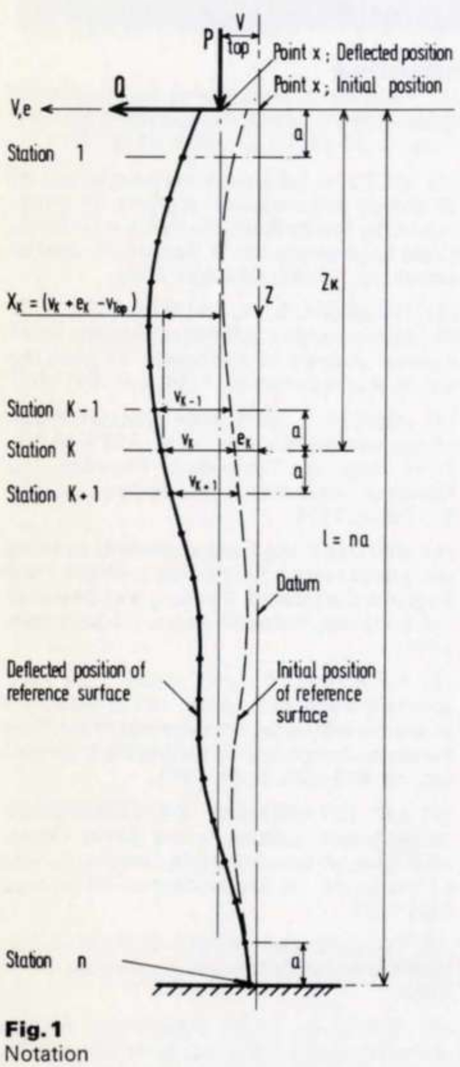

Fig. 1
Notation

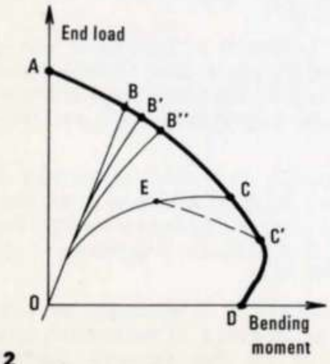

Fig. 2
Interaction curve

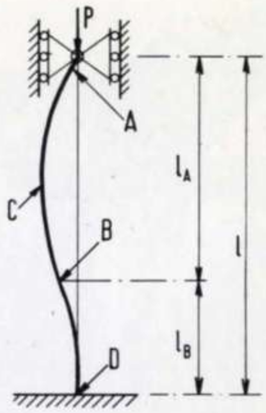

Fig. 3
Buckled column

Notation

- P is the compressive end load applied at the top of the column.
- Q is the lateral force applied at the top and it acts at right angles to P.
- M is the bending moment at a point in the column.
- X is a point in the top cross-section of the column through which P and Q act.
- e is the eccentricity of a reference surface in the column from a datum parallel with the proposed direction for P and passing through the initial (before column loading) position of point X; e defines the out of straightness.
- v is the sum of the lateral deflection from the initial cantilever deflection of the top of the column and that from application of P.
- v_{TOP} is the initial lateral deflection of the top of the column.
- z is the distance of a column section from the top of the column.
- v_R, z_R are the lateral deflection and distance respectively from the column top for a chosen principal section in the column.
- a is the length of a column segment.
- L is the length of the column.
- n is the number of segments in the column ($L = na$).
- y is the distance of a column fibre from the reference surface – measured positive in the positive v direction.
- ϵ, σ are the tensile strain and stress respectively for a fibre.
- ϵ_R is the strain at the reference surface.
- m, c are constants in the moment/curvature relationship.
- f_{cu_t} is the concrete cube strength at time t days.

Numerical analysis

Objective

The analysis should be able to compute the ultimate strength of the column, as in Fig. 1, taking into account an initial lateral deflection of the pin-joint at the top, the effect of lack of straightness and the changing effective cross-section of the column; the changes in effective cross-section result from taper in the column and from variation in cracking at different levels and at different stages of loading.

Assumptions

(i) Both the concrete and the steel are homogeneous.

(ii) Lateral deflections of the column are small compared with its length.

(iii) Shear distortions make a negligible contribution to lateral deflections.

(iv) Plane sections remain plane during bending.

(v) The concrete is unable to sustain tensile stresses.

(vi) The stress/strain properties of the column materials are known.

(vii) The stress/strain curves for the column materials are reversible.

Choice of stress-strain curves: steel

The steel will be assumed to behave identically in compression and tension, and, up to the yield stress, will be considered to have a perfectly linear relationship between stress and strain as defined by the modulus of elasticity. There will be no increase in stress with strain after the yield stress. If the material does not have a well defined yield stress then the stress corresponding to a .002 proof strain will be used.

Choice of stress-strain curves: concrete

The single equation proposed by Desayi and Krishnan[5] for the behaviour of concrete in compression will be used.

That is

$$\sigma = \frac{2\left(\frac{\epsilon}{\epsilon_m}\right)\sigma_m}{1+\left(\frac{\epsilon}{\epsilon_m}\right)^2} \qquad \text{———} \quad 2.$$

Where σ_m is the maximum stress achieved in the concrete and ϵ_m is the corresponding strain. Fig. 5 illustrates the resulting stress-strain curve.

The initial slope of the curve gives a modulus of elasticity

$$E_C = \frac{2\,\sigma_m}{\epsilon_m}$$

which agrees well with the measured performance of concrete. There is also good agreement in general with the Hognestad[10] stress strain block. Neogi, Sen and Chapman[3] used Equation 2 in their studies of eccentrically-loaded, concrete-filled, steel tubular columns.

The choice of values for σ_m and ϵ_m will now be considered. In determining the capacity of a concrete column, it is necessary to calculate the strength both when the load is first applied and when the column is fully cured. With a fully cured column the concrete strength will have risen, but the effect of sustained loading for a period of years will be to cause sustained creep strains. The lower strength should be regarded as the column capacity. Note that unless the first loading of the column is at an unusually young age the strength at several years will be the lower. In any case it is usually possible to argue that the first loading is only partial loading that the full load will not be applied for several weeks, by which time if the concrete was young at first loading, the strength will have improved substantially.

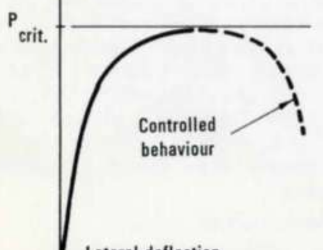

Fig. 4
Load deflection curve

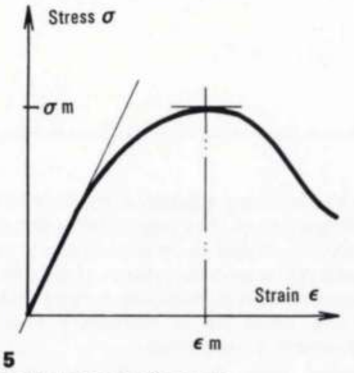

Fig. 5
Concrete stress-strain curve

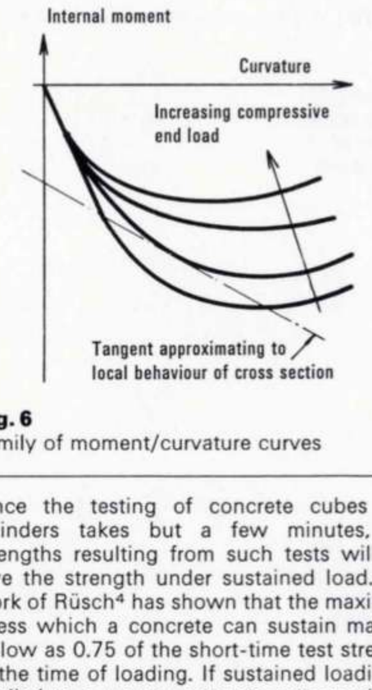

Fig. 6
Family of moment/curvature curves

Since the testing of concrete cubes and cylinders takes but a few minutes, the strengths resulting from such tests will not give the strength under sustained load. The work of Rüsch[4] has shown that the maximum stress which a concrete can sustain may be as low as 0.75 of the short-time test strength at the time of loading. If sustained loading is applied to a young concrete test specimen, then, owing to the increasing strength with curing, it is possible after a critical time to start gradually increasing the load. As an example, the critical time for a concrete 10 days old is approximately six hours after the first application of the sustained load.

In the light of the above discussion the following stress-strain curve parameters are suggested:

(i) For analysis of strength when loading first applied t days after casting:
$\sigma_m = {\cdot}75 f_{cut}$ $\quad \epsilon_m = {\cdot}0025$

(ii) For analysis of strength under sustained load when concrete fully cured:
$\sigma_m = 1{\cdot}05 f_{cu28}$ $\quad \epsilon_m = {\cdot}0060$

Note that there is a ratio of 2.4 between the peak stress strains for the two cases and this may seem conservative compared with the suggestion by Broms and Viest[11] of stretching the short-time stress strain curve by a factor of 2.0 in the strain direction to allow for long term effects. However, Mauch[1] (Paper 13), while reporting his work on the effects of creep and shrinkage on the capacity of concrete columns, suggests that the factor of 2.0 may not be large enough.

Representations of cross-sections

Sections of the column will be divided into several strips at right angles to the plane of column deflections (i.e. parallel to the reference surface). The distance of the centroid of each strip from the reference surface axis will be calculated and input to the computer, together with the areas of steel and concrete in the strip. Should a reinforcing bar not be at the centroid of the concrete area in the strip, then the area of the steel of that bar will be shared between the two adjacent strips in proportion to the distance of the bar centre from the centroids of the concrete strips. The accuracy of the representation will depend on the number of strips taken. For most sections, 10 to 15 strips will give accuracies for second equivalent moments of area better than $\pm 1\%$. More than 15 strips is hardly justified, unless there is some way of guaranteeing that the position of the reinforcing will be within a very small tolerance of the position given on the drawings.

Establishing the moment equilibrium equations

The column is divided into n segments of equal length a. Between 15 and 20 segments will give sufficient accuracy for engineering purposes. The junctions between segments and the junction between the final segment and the base will be referred to as stations, and are numbered consecutively from 1 to n down the column.

A reference surface is chosen for the column. This is a surface within the column, preferably close to the centre, running the length of the column and at right angles to the plane of buckling. The cross-sectional shapes at all stations are defined relative to it. The eccentricity e of this surface describes its distance before loading from a line through the top section point X and parallel to the proposed line of action of the end load P; allowance for out-of-straightness is made in the value assigned to e at each station.

The analysis will take as a starting point the bending equilibrium equations for each station. The external moments of the forces P and Q about the axis, formed by the intersection of the reference surface with the station cross-section, is equated to the moment of the internal stresses about the same axis.

If the distortions of the column are within the linear range of the column materials, then it is possible to simply write the internal moment as $-EI^{d^2v}/_{dz^2}$ where $-^{d^2v}/_{dz^2}$ is the local curvature. Outside the linear range it is possible at a given end load to express the internal bending moment as linearly increasing with curvature, provided only small changes of curvature are involved; the local tangent to the M against $^{d^2v}/_{dz^2}$ curve for the particular end load could be used to do this. Fig. 6 illustrates a family of moment/curvature curves for a particular cross-section.

Any local tangent can be described by the straight line equation

$$M = m\frac{d^2v}{dz^2} + C \qquad \text{———} \quad 3.$$

where m is the slope of the tangent and C its intercept with the M axis.

The bending equilibrium equation at any station K on the column now becomes

$$P(v_K + e_K - v_{TOP}) - Qz_K$$
$$= m_K\left(\frac{d^2v}{dz^2}\right)_K + C_K \qquad \text{———} \quad 4.$$

The finite difference form for the curvature is

$$\left(\frac{d^2v}{dz^2}\right)_K = \left(\frac{v_{K-1} - 2v_K + v_{K+1}}{a^2}\right)$$

Now when $K = 1$ (at the first station)

$$V_{K-1} = V_{TOP}$$

and when $K = n$ (at the base station)

$$\left.\begin{aligned} V_K &= O \\ \text{and} \quad V_{K+1} &= V_{K-1} \end{aligned}\right\} \text{Complete lateral and rotational restraint.}$$

Thus the set of equilibrium equations for the n stations will appear as

$$
\begin{aligned}
P(v_1+e_1-v_{TOP}) \quad - Qz_1 &= \frac{m_1}{a^2}(v_{TOP}-2v_1+v_2) + C_1 \\
P(v_2+e_2-v_{TOP}) \quad - Qz_2 &= \frac{m_2}{a^2}(v_1-2v_2+v_3) + C_2 \\
P(v_{n-2}+e_{n-2}-v_{TOP}) - Qz_{n-2} &= \frac{m_{n-2}}{a^2}(v_{n-3}-2v_{n-2}+v_{n-1}) + C_{n-2} \\
P(v_{n-1}+e_{n-1}-v_{TOP}) - Qz_{n-1} &= \frac{m_{n-1}}{a^2}(v_{n-2}-2v_{n-1}+o) + C_{n-1} \\
P(o+e_n-v_{TOP}) \quad - Qz_n &= \frac{m_n}{a^2}(v_{n-1}+o+v_{n-1}) + C_n
\end{aligned}
\tag{5}
$$

Solution of the equations

The process for determining the tangents to the moment/curvature curves, and hence values for m and c at each station, is outlined in Appendix 1. It can be seen that the n simultaneous equations 5 contain $n+1$ unknowns which are

$P, Q, V_1, V_2, \ldots\ldots\ldots\ldots, V_{n-1}$.

The solution must therefore be a step-by-step one taking the form of assigning a value to one of the unknowns, solving the equations for the other n unknowns and then repeating the procedure as many times as is desired, each time with a new assigned value.

Two approaches seem possible:

(i) To examine the behaviour of the column with varying P, i.e. to assign a value to P and solve the equations

(ii) To examine the behaviour of the column with variation in lateral deflection at one station which will be called the principal station, i.e. to assign a value of lateral deflection at the principal station and to solve the equations.

The first approach is not satisfactory. From Fig. 4 it can be seen that as the peak end load is approached, two deflected positions are theoretically possible for any given end load and the closer to the peak load, the closer are the deflected positions. The equations become very poorly conditioned and at the peak load the matrix of coefficients of the unknowns will be singular.

The second approach does not have this problem, provided the principal station is chosen sensibly. For the best results it should be the station where lateral deflections are on average expected to increase most rapidly. If a bad choice is made, it may happen that, with increasing compressive end load, there is at first say, an increasing deflection and later a decreasing deflection, which will result in two possible end loads for a given principal station deflection. Problems!

The chosen station will be called station R and so V_R is a known quantity. There are now as many simultaneous equations as there are unknowns; a series of calculations will be performed, each with an increase in V_R until no further significant increase in end load is observed. The behaviour of the column beyond the peak load is not important for the purpose of this investigation and will not be examined in the computer program.

The simultaneous equations as they stand are non-linear, in that the variable P multiplies the variables

$V_1, V_2 \ldots\ldots, V_{R-1}, V_{R+1}, \ldots\ldots, V_{n-1}$.

A procedure must be adopted to linearize the equations by substituting realistic guessed values of lateral deflection to suit the chosen V_R into all the brackets multiplying P and to solve the resulting equations for P, Q and the lateral deflections. The lateral deflections from the solution should be an improvement on the guessed values and the process of substitution of guessed values and solution may be repeated as many times as is necessary until the improvement is very small.

In matrix form the equilibrium equations become

$$[Y]\,[U] = [X] \tag{6}$$

Where [U] is the column matrix of unknowns

$$
\underset{(n\times 1)}{[U]} = \begin{matrix} V_1 \\ V_2 \\ V_3 \\ \\ V_{R-1} \\ V_{R+1} \\ \\ V_{n-1} \\ P \\ Q \end{matrix}
\tag{7}
$$

Where [X] is the column matrix of constants

$$
\underset{(n\times 1)}{[X]} = \begin{matrix} \left(C_1+\frac{m_1}{a^2}V_{TOP}\right) \\ C_2 \\ C_3 \\ \\ \left(C_{R-1}+\frac{m_{R-1}}{a^2}V_R\right) \\ \left(C_R - \frac{2m_R}{a^2}V_R\right) \\ \left(C_{R+1}+\frac{m_{R+1}}{a^2}V_R\right) \\ \\ C_{n-2} \\ C_{n-1} \\ C_n \end{matrix}
\tag{8}
$$

Where [Y] is the un-symmetric square matrix of coefficients (see Fig. 7).

Solution of these equations will be by a form of elimination which involves interchanging rows of Y and X to keep the largest element of the current column of Y on the leading diagonal. This technique improves the accuracy of the solution when the equations are poorly conditioned and is explained in detail by Livesley[9], chapter 2.6.

Sample application

Introduction

In the design of the new restaurant building for the University College Dublin campus at Belfield, Ove Arup & Partners (Dublin) were faced with the problem of determining accurately the capacity of four slender reinforced concrete columns supporting the umbrella type roof. Codes of practice could not be used with any satisfactory degree of confidence in this case and the research outlined in this paper was undertaken to provide a solution. Without going into the details of the structural considerations involved in defining the problem, it was concluded that a precast column, having the cross-section given in Fig. 8, should be examined as 11.15 m long, built in at the base, pinned at the top and with vertical load applied through the centroid of the equivalent section at the top. The column was to sustain a design vertical load of 520 kN. Thermal effects could produce a movement at the top in the plane of symmetry of the section of ± 9.5 mm. High quality precasting gave an out-of-straightness of ± 3.2 mm.

For the purpose of the computer analysis, the column was divided into 20 equal length segments and cross-sections were represented by 15 slices. The principal station was chosen as Station no. 8, numbering from the top downwards. Analysis by the computer program included analyses with and without the tapering steel plates, short and long term calculations being performed in each case. Since they taper, the addition of the steel plates automatically introduces eccentricities for the vertical load, regardless of any out-of-straightness.

Lowest modes of buckling

Since double curvature is present, the lowest mode of buckling is produced when the top $\frac{2}{3}$ of the column has its lowest bending stiffness after due allowance has been made for cracking of the concrete. Consideration of the pure moment, cracked section, equivalent second moments of area indicated that, without the steel plates, the outer heel of the top portion of the column would be in compression for the lowest mode, while with the steel plates it would be in tension. Consequently, for the purpose of the analyses, the direction of the 9.5 mm movement at the top and the 3.2 mm maximum out-of-straightness were chosen to encourage the lowest modes.

Assumed material properties

(i) Reinforcing bars

$E = 2{\cdot}07\times10^5$ N/mm²

Yield strain = ·0022

(Note that these assumed figures imply a yield stress of 455 N/mm² irrespective of bar size).

(ii) Steel plates

$E = 2{\cdot}07\times10^5$ N/mm²

Yield strain = ·0012

Yield stress of 248 N/mm²

(iii) Concrete

Grade 75 concrete was specified. Cube tests indicated strengths at 28 days greater than or equal to 51.7 N/mm². For the purpose of analysis $f_{cu_{28}} = 51.7$ N/mm² will be taken. The short term analysis will determine the capacity at 56 days after casting, when the cube strength will be roughly 1.15 times the 28 day strength. Thus in the calculation of σ_m for the short term analysis, .75 $f_{cu_{56}}$ approximates to .85 $f_{cu_{28}}$. The following values are therefore taken for the parameters in Equation 2:

Short term analysis

$$\begin{cases} \sigma_m = \cdot 85 f_{cu_{28}} = 43{\cdot}9 \text{ N/mm}^2 \\ \epsilon_m = \cdot 0025 \end{cases}$$

Sustained loading, fully cured analysis

$$\begin{cases} \sigma_m = 1{\cdot}05 f_{cu_{28}} = 54{\cdot}2 \text{ N/mm}^2 \\ \epsilon_m = \cdot 0060 \end{cases}$$

$[Y] = (n \times n)$

$+\frac{2m_1}{a^2}$	$-\frac{m_1}{a^2}$	○	○	○	○	○	○	○	○	○	○	○	○	x_1	$-a$
$-\frac{m_2}{a^2}$	$+\frac{2m_2}{a^2}$	$-\frac{m_2}{a^2}$	○										○	x_2	$-2a$
○	$-\frac{m_3}{a^2}$	$+\frac{2m_3}{a^2}$	$-\frac{m_3}{a^2}$	○									○	x_3	$-3a$
○	○	•	•	•	○								○		
○		○	•	•	•	○							○		
○			○	•	•	•	○						○	x_{R-2}	$-(R-2)a$
○				○	$-\frac{m_{R-1}}{a^2}$	$+\frac{2m_{R-1}}{a^2}$	○						○	x_{R-1}	$-(R-1)a$
○					○	$-\frac{m_R}{a^2}$	$-\frac{m_R}{a^2}$	○					○	x_R	$-Ra$
○						○	$+\frac{2m_{R+1}}{a^2}$	$-\frac{m_{R+1}}{a^2}$	○				○	x_{R+1}	$-(R+1)a$
○						○	•	•	•	○			○	x_{R+2}	$-(R+2)a$
○							○	•	•	•	○		○		
○								○	•	•	•	○	○		
○									○	•	•	•	○		
○										○	$-\frac{m_{n-2}}{a^2}$	$+\frac{2m_{n-2}}{a^2}$	$-\frac{m_{n-2}}{a^2}$	x_{n-2}	$-(n-2)a$
○											○	$-\frac{m_{n-1}}{a^2}$	$+\frac{2m_{n-1}}{a^2}$	x_{n-1}	$-(n-1)a$
○	○	○	○	○	○	○	○	○	○	○	○	○	$-\frac{2m_n}{a^2}$	x_n	$-na$

This particular example of [Y] corresponds to $n=16$, $R=8$. i.e. 16 column segments and the principal station at the junction between the 8th. and 9th. segments.

Where $x_K = (v_K + e_K - v_{\text{TOP}})$ —————— 9.

Where K has values 1, ... R, ... n

Fig. 7
The matrix of coefficients for Equation 6.

Results of analyses

Fig. 9 shows the variation in column end load with lateral deflection at the principal station (i.e. at 4.39 m from the top) for the four analyses performed. The non-zero deflections for each case at zero end load are due to the cantilever action of the column to provide the initial 9.5 mm deflection at the top. In all cases the curves are still rising when the decision was made within the computer program to stop calculations. Clearly the increases in the capacities beyond the termination points will be small.

The capacity of the column is 1700 kN without the plates and 2470 kN with the plates.

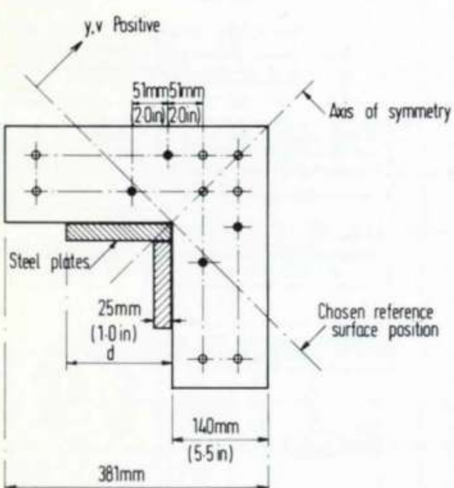

Fig. 8
Column cross-section

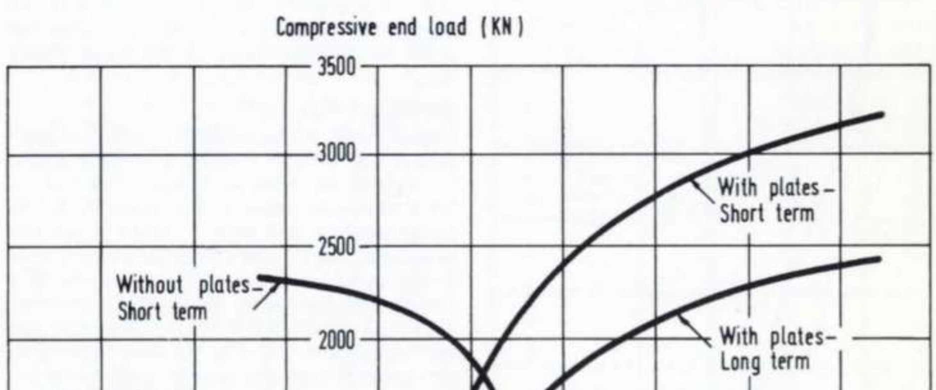

Fig. 9
Deflections at Station no. 8

The deflected shape of the column at the 1700 kN load is shown in Fig. 10.

Conclusions

A method of computer analysis of slender reinforced concrete columns has been developed in which all the significant parameters governing the capacity of the columns have been allowed for. The details of the application of the method to the particular case of columns built in at one end and pinned at the other have been given. Other types of end conditions can be catered for by suitable modifications to Equation 5.

The long-term strength is the criterion of the ultimate capacity. If the design load is only a fraction of the long-term capacity then it would perhaps be misleading to say that, if the load were increased to the capacity load, the column would then fail. It is only if the capacity load is held constant for a period of years, so that creep deflections can develop fully, that failure will occur.

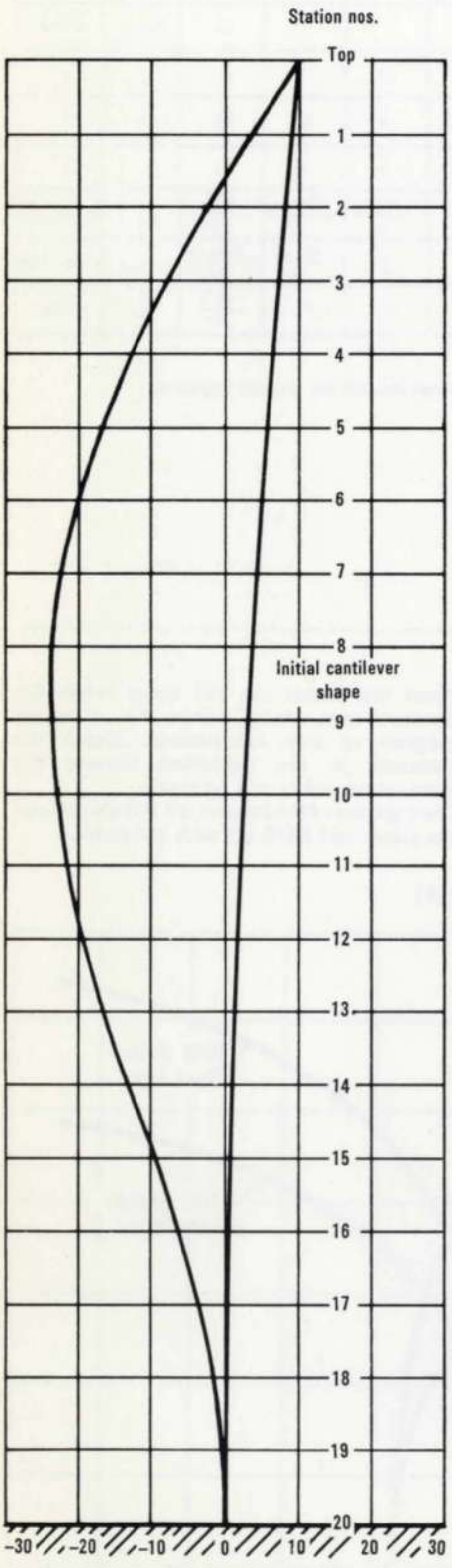

Fig. 10
Deflected shape

Appendix 1 – Computer program

The main program flow diagram, Fig. 11, outlines the numerical procedures used to achieve the solution. Some of the procedures are discussed in the following paragraphs.

Guesses for the deflected shape and P

In the following notes the calculations corresponding to any given value of V_R are referred to as a stage.

There is a preliminary stage called the Initial Cantilever Stage in which the deflections at all stations, including the principal station, are calculated corresponding to a specified column top deflection V_{TOP}, caused by a lateral load Q alone ($P=O$) being applied at the top. One solution only of Equation 5 is involved in which it is assumed that the column material obeys a linear elastic law everywhere so that $m=EI$ and $C=O$ in Equation 3. Equation 5 must be modified to take account of a new unknown V_R and the removal of another P ($=O$). Subroutine SPESHL performs these calculations.

The first stage follows on from the Initial Cantilever Stage and, for the first solution of Equation 5, the engineer chooses an increment in the deflection V_R of the principal station from the Initial Cantilever Stage value then uses his judgement to predict the corresponding changes in deflection at the other stations along the column. For the first solution it will again be assumed that $m=EI$ and $C=O$ and consequently the moment arm for P must be changed in this case to be the distance to the centroidal axis of the uncracked equivalent section and not (as for all subsequent solutions) the distance to the chosen reference surface. Following the first solution, solution values are automatically applied as improved predictions for a second solution and so until no further significant change in the end load is observed.

The second and subsequent stages are completely automatic. Principal station deflections are first incremented by the amount initially chosen by the engineer. Guesses for the lateral deflections at all the other stations, and for the end load, will be the previous stage finalized values plus the increment of these values over those for the stage before that.

Evaluation of m and C

Evaluation of the parameters m and C at each station will involve making a realistic guess, not only at the deflected shape of the column for a particular value of V_R, but also at the corresponding end load P. Once a realistic guess has been made at the deflected shape, a first approximation to the curvature at a station can be made. Using this curvature, the reference surface strain at the station can be adjusted until the internal end load corresponds with the realistic guess, and the internal moment of resistance for this station can then be evaluated.

The curvature can now be increased by a small increment and the process of reference surface strain adjustment should be repeated to give the guessed end load. A new value of internal moment will be calculated. It is now possible with the two known curvatures and moments to construct the straight line which approximates to the tangent to the moment/curvature curve for the guessed P; thus m and C are evaluated.

Convergence

The computer program involves solving the structure for a series of principal station deflections until no significant change in the end load with deflection is observed. The choice of an acceptably small change in end

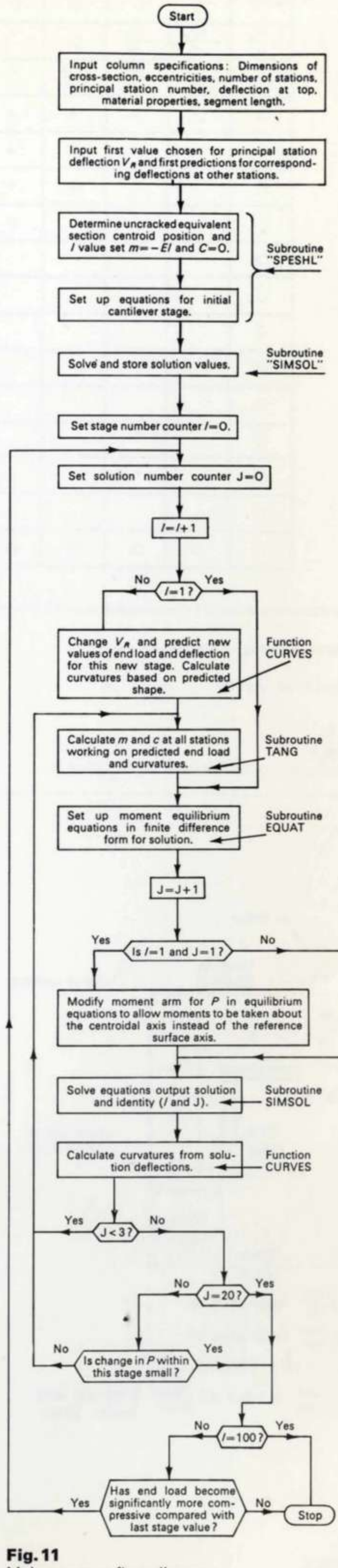

Fig. 11
Main program flow diagram

load between stages, before the calculations may be terminated, has a substantial influence on the number of stages necessary. If the acceptable change is large, then the capacity of the column may be underestimated. On the other hand if the acceptable change is small, a large number of stages could be necessary, most of the later stages yielding very little additional information.

The size of the principal station deflection increment between stages is critical to the functioning of the program. The number of stages necessary to achieve a particular end load is inversely proportional to the size of the increment. Yielding to the temptation to take a large increment could cause difficulty with the accuracy of the predictions for the corresponding end load and deflected shape. This in turn could either cause divergence in the iterations of the subroutine TANG and consequent divergence in the solutions for the stage, or direct divergence in the solutions for the stage.

Subroutine TANG is at the heart of the program and involves changes in the guessed reference strains in order to achieve an internal end load P close to the guessed external end load PG. The most satisfactory technique with predictions of *EPREF* is to start as close to the top of the column as possible, at Station no. 1, where the curvature strains have their smallest influence and to take the first guess at the strain achieved there in the previous solution. When moving on to the next station down the column, the best initial guess for *EPREF* will be the finalized value for the previous station. Modification in *EPREF* at any station is critical; too large a change or too small a change may mean a large number of iterations. The precision with which P should approximate to PG also has a strong influence on the required number of iterations but it also influences the stability of the solutions within each stage. Should P only be required to be within $\pm 5\%$ of PG, then it may cause divergence, since m and C are exceedingly sensitive to end load except during the early stages.

Appendix 2 – Subroutines

SPESHL

The Initial Cantilever Stage sets up most of the conditions for the main program to function in a routine manner. Subroutine SPESHL, apart from preparing for the solution of the Initial Cantilever Stage, assigns values to certain variables in preparation for the first solution of the first stage. The flow diagram for SPESHL is given in Fig. 12.

The equilibrium equations for the Initial Cantilever Stage in matrix form are:

$\frac{2m_1}{a^2}$	$-\frac{m_1}{a^2}$	O	O					O	$-a$	v_1	$\frac{m_1 v_{TOP}}{a^2}$
$-\frac{m_2}{a^2}$	$\frac{2m_2}{a^2}$	$-\frac{m_2}{a^2}$	O					O	$-2a$	v_2	O
O	$-\frac{m_3}{a^2}$	$\frac{2m_3}{a^2}$	$-\frac{m_3}{a^2}$					O	$-3a$	v_3	O
		•	• •	• • •	• •	•					=
O					$-\frac{m_{n-3}}{a^2}$	$\frac{2m_{n-3}}{a^2}$	$-\frac{m_{n-3}}{a^2}$	O	$-(n-3)a$		
O					O	$-\frac{m_{n-2}}{a^2}$	$\frac{2m_{n-2}}{a^2}$	$-\frac{m_{n-2}}{a^2}$	$-(n-2)a$		
O					O	O	$-\frac{m_{n-1}}{a^2}$	$\frac{2m_{n-1}}{a^2}$	$-(n-1)a$	v_{n-1}	O
O					O	O	O	$-\frac{2m_n}{a^2}$	$-na$	Q	O

$(n \times n)$ $(n \times 1)$ $(n \times 1)$

TANG

The purpose of subroutine TANG is to establish at each station K the parameters m_K and c_K defining the tangent to the moment-curvature curve for the guessed end load PG and the guessed curvature *CURVAT*. The 'tangent' is found by considering two points very close together on the curve and establishing the equation of the straight line passing through them. One point will correspond to *CURVAT* and the second point to *CURVAT* plus a standard increment in curvature *DCURVE*.

Fig. 13 shows an outline of the flow diagram.

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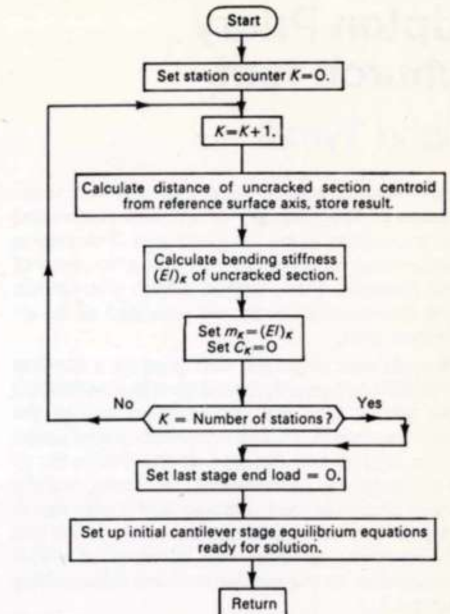


Fig. 12
Outline flow diagram for SPESHL

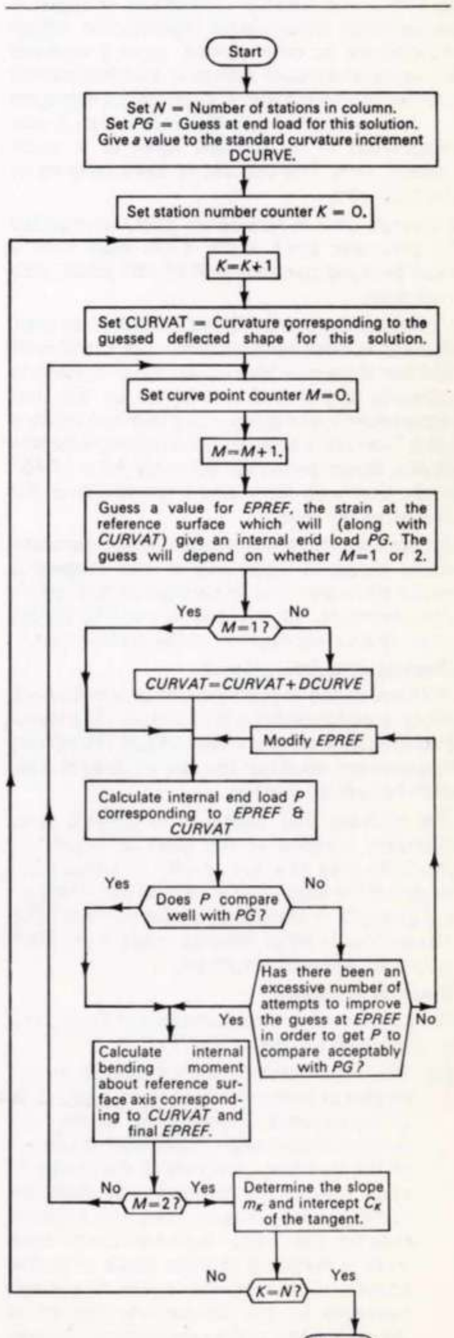


Fig. 13
Outline flow diagram for subroutine TANG

Upton Priory church roof

John Tyrrell

Upton Priory has now become part of the conurbation of Macclesfield which was nominated for expansion some 20 years ago. The area is designated for development along the lines of the Radburn Plan, within which the church and community centre are intended to be an integral part.

The site was originally occupied by a number of buildings but only an old barn has withstood the test of time and the effectiveness of the local vandalism. This barn was renovated seven years ago to the designs of the Professor of Architecture at Manchester University, but the rustic character was retained and it was desirable for any new buildings to blend in with this environment to offer any glimmer of visual satisfaction to the residents of the surrounding estates.

We first become involved in the schemework for the church and community centre as early as 1967. The Church Authorities engaged a professional fund-raising organization which replenished its own coffers, gave a nominal amount to the church authority, and then quietly withdrew from the scene. As a result, the project lay dormant until early last year when it was resurrected in its present form at a much reduced cost. The present scheme consists of the following:

a church, 20.1 m square on plan, surrounded by perimeter brick walls 3.7 m high with a square-based pyramid roof of 45° pitch, clad with slate;

a community centre, 13.7 m by 22.9 m on plan, also surrounded by a perimeter brick wall with additional internal block piers for intermediate supports to the roof as defined by the user requirements, and connecting with the existing barn. The roof is a combination of two adjacent square-based pyramids, 9.1 m by 9.1 m of 45° pitch, clad with slate, and a conventional flat roof at eaves level;

the community centre is built up from domestic scale structural units and in this respect is straightforward. The remainder of this article will, therefore, be devoted to the structurally more interesting aspects of the church roof.

Design considerations

We were asked to produce a structure forming a four-sided pyramid and which would provide an enclosed volume of similar shape. The second requirement ruled out the use of deep trusses with horizontal booms.

The conventional approach of having ridge members meeting at the apex as a primary structure was rejected, partly to reduce the length of the individual elements and partly on the grounds of aesthetics, as the minimal costs allowed for internal finishes made it desirable to use an exposed structure.

Roof structures

The timber roof structure consists of three parts, as illustrated in Figs. 2 and 3:

(a) The primary structure is an arrangement of trusses of which there are two for each side of the pyramid. These trusses spring from reinforced concrete columns at the corners of the building, intersect at the centre of each side of the pyramid and meet the corresponding trusses from the adjacent side on the ridge between them. Each truss is made up of three levels of timber spaced apart vertically so as to provide passages for the intersecting timbers at the centre and at the same time to increase the effect of depth.

Each level of timber in the main truss contains three members separated by thin plywood webs which form open castellations

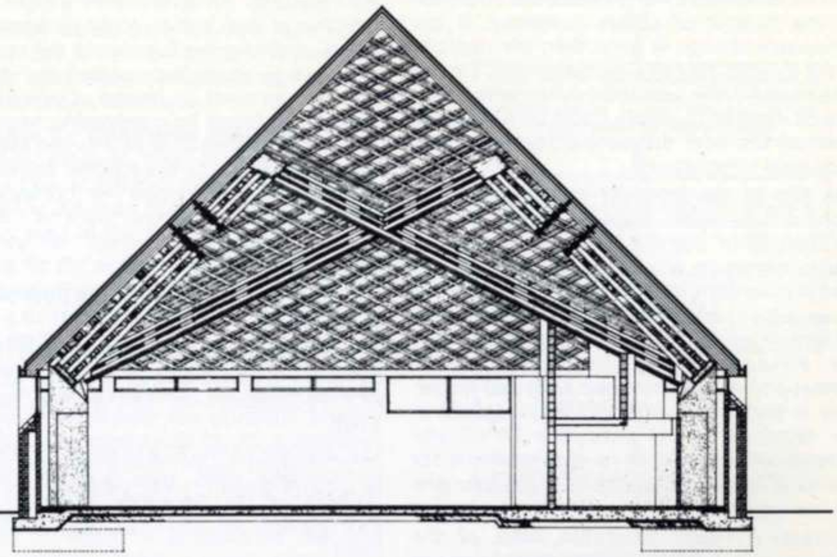


Fig. 1
Section. (Reproduced by courtesy of the architects)

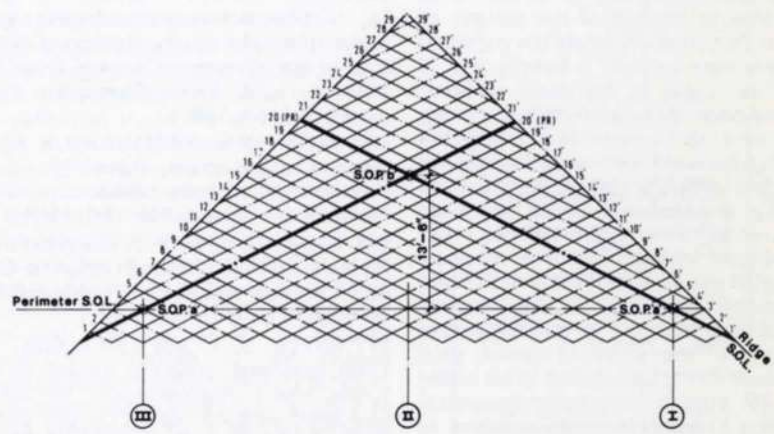


Fig. 2a
Vertical elevation on setting out grid

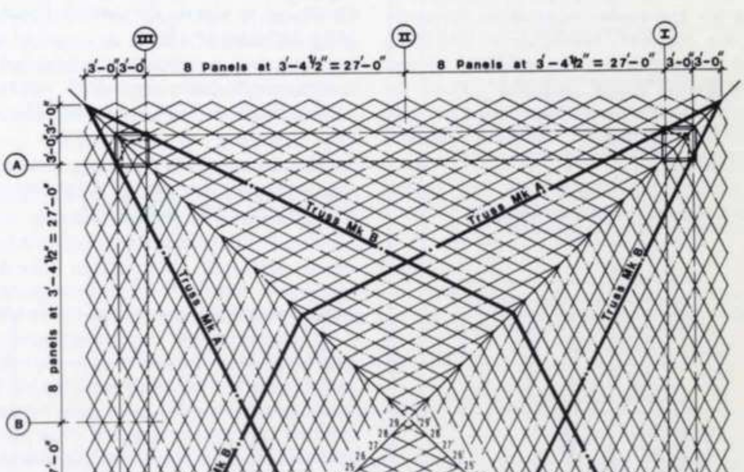


Fig. 2b
Plan on setting out grid

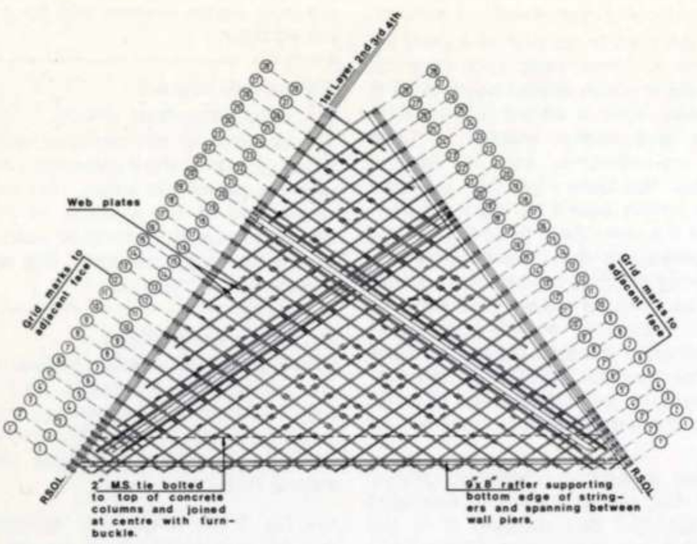


Fig. 3
Plan on stringers, first and second layers shown

in the plane of the truss and extend above the top member to connect to the secondary structure. The plywood plates are in pairs to increase the contact area and to ease the connection problems as well as to provide a torsional restraint to the trusses, as when placed in position their planes are inclined from the vertical and they would thus be twisted due to their own weight.

(b) A secondary system of four layers of intersecting stringers provide a rhomboidal grillage where each rib is castellated. The twin stringers in any direction are joined by plywood plates across the intervening space to increase the effective shear strengths and to eliminate reliance upon pull-out resistance of nails as the only means of holding the layers together.

(c) The top structural layer of tongued and grooved boarding running up the roof plane triangulates the rhombes of the secondary structure to provide a system with in-plane resistance.

Load transfer through structure

Loads normal to the roof plane are carried by the secondary system in flexure. The stringers are mutually supporting along the ridge, supported on a wall plate along the eaves perimeter and supported on the primary trusses through the ply-wood web plates. The trusses are propping each other at the ridge intersection and spanning in flexure. The axial component of the load in the trusses is transferred to the reinforced concrete columns which are tied to each other around the perimeter.

The secondary stringers form an in-plane bipod. The in-plane loads are transferred to the bipod by the triangulated boarding and secondary stringer system. The bipod thrusts are transferred through ply webs to the primary trusses and from there into the columns and tie-bar arrangement.

Columns

Figs. 5 and 6 illustrate the developed form of the column heads which are designed to provide a solid thrust block to the primary trusses. The column profile changes at the lower cill level along the perimeter walls to an equal sided 'L' shape.

The square-cut ends of the primary trusses are fitted with galvanized mild steel shoes which are bolted to the column head. The columns are tied together at their heads along the perimeter by 50 mm diameter tie bar, bolted to the column below the soffit of the lowest layer of stringers and joined mid-way between the columns by a turnbuckle.

Fig. 4
Truss intersections. (Photo: John Tyrrell)

Erection

The primary structure was erected by constructing a staging platform to provide a temporary seating to the trusses. The prefabricated roof trusses were delivered to site with the plywood web plates above the 'finger' intersection only temporarily fixed for rigidity during transportation. These were removed prior to erection to allow for freedom of movement during the intermeshing of the trusses.

The trusses were lifted individually by crane and carefully threaded through their neighbours. Although some difficulty was experienced during this procedure, partly due to the flexibility of the trusses about their minor axes and partly due to small variations in the column head, the operation was carried out satisfactorily.

Because of the need for temporary support of the secondary structure during erection, the staging platform was extended to provide a birdcage network within the contained volume.

Connections

All timber to timber joints were designed as straightforward nailed connections. This was largely because of the numerous joints involved, particularly in the secondary structure, and the ease with which nailing can be carried out.

Within each roof panel all the stringers were butt-jointed between the web plates in a predetermined pattern to ensure continuity of stresses and to allow for the lengths of timber available.

Each line of longitudinal members of the primary trusses was cut and fixed at the ridge lap intersection, packed with diamond-shaped ply wood pieces and nailed from both faces.

Figs. 5 & 6
Column head. (Photo: John Tyrrell)

Cost

Once the general structural principles are understood, the construction is relatively straightforward. One of our problems was to communicate this to the contractor in order to obtain a competitive price. The contractors invited to tender for the job were selected on a regional basis and were, by national standards, relatively small in size.

To overcome this, John Harvey, who was largely responsible for the conception of the roof scheme, made up a large scale model which was exhibited and explained to each of the contractors during the tendering period. The success of this operation was reflected in the contract value which is about £60,000.

Conclusions

It was originally hoped that the construction would be largely completed before publishing this article but unfortunately this is not the case. However, the structure is illustrated, if imperfectly, by the photographs that were taken during the various stages of construction. The church should be completed and consecrated early in the new year.

Credits

Client: Diocese of Chester
Architect: Garner Preston & Strebel
Quantity surveyors: David S. Channon & Partners
Contractor: Isaac Massey & Sons
Truss manufacturers: Walter Holme & Sons Ltd.

Dilating clay equations

Alistair Day

Statement of the problem

As in other branches of engineering, soil mechanics calculations are made to establish the adequacy of a design, and the analytical tools used will be those available at the time the design is being made. In soil mechanics there is the additional complication that there are two parts to a design analysis; the numerical computation and the experimental derivation of data. These two will not necessarily advance together, so that at any stage there can be computational methods available which require data which is not readily available or cannot be obtained from existing equipment. Alternatively, data may be available which cannot be used in calculations. The latter appears to be the case for overconsolidated clays when stress paths obtained experimentally show their dilating and brittle nature, but this information does not appear to be used in computational methods which have been published.

The object of the work, part of which is described in this paper, was to try to develop a program which could use as much of the available experimental data as possible.

There are essentially two requirements for this. First, establishing a set of equations which can adequately represent the observed behaviour of soil samples in experimental equipment, and, secondly, the development of a program which will use the equations and also reproduce the structure as a numerical model. In both these parts, it may be that restrictions are imposed on the generality of the program so that it will only deal with the types of calculation likely to be required for design work. When a program can produce some of the desired calculations it may be possible to extend it to incorporate other desirable features.

There are two ways of approaching a problem. One is to attempt to build up a theoretical model which, if accurate, will reproduce the observed data. The alternative is developing empirical relationships which give the desired results. In effect, the latter approach was tried first but did not prove very satisfactory. The best way was found to be a combination in which a set of equations was developed which was able to represent the general behaviour of overconsolidated clays and then provide as many coefficients as necessary to obtain as close a fit to experimental curves as desired.

Two types of calculation are envisaged:

(i) Deflection at working load

(ii) Load factor or stability.

These calculations are likely to be required for both immediate and long term conditions.

In the load factors type of calculation for an excavation or embankment, the stresses will always start from some given initial condition and increase (or decrease) to the final stresses, but this may not always be the case for calculating the deflections at various stages of construction. However, for the time being, the stresses will be presumed to change in one direction as this significantly reduces the amount of experimental data required.

The form the equations will take can be governed by the method of solution used and this can prevent the equations including effects which are necessary for a complete description of all the soil behaviour. If no limitations are imposed, then it should be possible to build up a set of equations by examining and providing for each of the parameters which appear to be required. This is the approach which is adopted here to derive a set of constitutive equations for clay.

The clay is assumed to consist of a skeleton of soil particles and pore water, each of which will require stress-strain relationships. At first an attempt was made to derive functions for the stresses and strains related to fixed rectangular co-ordinates, but it became apparent that functions for the principal stresses and strains would be more satisfactory because the non-linear behaviour of the material depends on the principal stresses. In the following sections the set of equations derived can be used for either plane stress or plane strain, depending on the coefficients used. The principal stresses are the in-plane stresses. When used for plane stress the out-of-plane strain is not determined and for plane strain the out-of-plane stress is not determined.

Fig. 1 shows a typical, idealized, general stress path for plane strain of saturated isotropic clays. On this diagram C is the consolidation pressure to which the clay has been loaded. If the soil were under equal total stresses at point C and one of the stresses is increased, then the effective stresses will follow the path CM. The concave downward form of the curve indicates that the soil is contracting when its stresses move away from point C. Any other stress paths which start at points between CD and beyond, will have a similar shape to CM and soil with stress paths in this region are found to fail plastically. If a soil has been consolidated by equal stress to point C and then unloaded to stresses appropriate to point E, then it is overconsolidated. If one of the stresses is now increased, the stress path EKF will be followed. In this case the concave upwards shape indicates that the soil is dilating. This soil fails in a brittle manner. There is a gradual transition of the shape of the stress path between the typical curve EKF near the origin to the curve CM. The object of the work described here was to establish a set of equations which could reproduce all the various stress paths in Fig. 1, together with the corresponding strain.

Nomenclature

In the following section the nomenclature used is:

$\sigma'_1\ \sigma'_2$ = principal effective stresses
p = pore pressure
σ'_a = mean of principal effective stresses
σ'_d = mean effective stress due to dilation
$\sigma'_e = \sigma'_a - \sigma'_d$
τ' = deviator effective stress
τ'_n = specified values of τ'
σ'_{ao} = initial value of σ'_a
σ'_n = specified values of σ'_a
ϵ_v = volumetric strain
γ = shear strain
v = specific volume
β = index for compression or decompression
κ = index for compression
λ = index for decompression
$l\ m\ n$ = integers
E = stress/coaxial strain (Young's Modulus)
ν = transverse strain/longitudinal strain (Poisson's Ratio)
D = mean stress/shear strain
$D_n E_n G_n H_n K_n$ = coefficients

Development of equations

Pore water pressure

Water has no shear strength, so it develops no stresses when subjected to shear strains. On the other hand hydrostatic stresses occur when it is subjected to volumetric strains, so the pore water stresses will be governed by the equation:

$$p=f(\epsilon_v) \qquad 1.$$

Soil particle stresses

(a) Volumetric strain effects

The skeleton of soil particles will develop a stress, similar to the hydrostatic pore pressure, under a volumetric strain. This stress can be considered as the average of the principal effective stresses and will be used in this way in the following sections. The equation for this stress will then be:

$$\sigma'_a=f(\epsilon_v) \qquad 2.$$

(b) Shear strain effects

When subjected to shear strains the particles can develop shear stresses. As functions for principal stresses are required, shear stresses will be considered in the form of the difference of principal effective stresses (the deviator stress) then:

$$\tau'=f(\gamma)$$

On Fig. 1 the offset from the diagonal OD, e.g. M' is proportional to the deviator stress. It can be seen that the maximum deviator stress limited by the locus SLR varies with the distance along the diagonal, i.e. with σ'_a, so it appears that the deviator stress at any shear strain (γ) is also dependent on the mean stress σ'_a. To include this effect the expression for τ' must be extended thus:

$$\tau'=f(\gamma, \sigma'_a) \qquad 3.$$

(c) Dilation and contraction

It is found that when soil is sheared, it tends to change its volume, dilating if it is an overconsolidated clay and contracting if it is a normally consolidated clay. Sands and other material can also show this property. If the material did not dilate or consolidate, then its undrained stress path would be, for example, CN in Fig. 1. The deviation of the stress path CM from the normal CN is a measure of the dilation/contraction and it is effectively a change of the mean stress with shear strain. It can be seen on Fig. 1 that this effect is also dependent on the mean stress as the curve is concave up at low (overconsolidated) mean stress and concave down at high (normal consolidated) mean stress. To allow for these effects, Equation 2 for mean stress must be modified to:

$$\sigma'_a=f(\epsilon_v)+f(\gamma, \sigma'_a) \qquad 4.$$

The equation is written in this form as the material behaves in a way which can most easily be explained by the mean stress consisting of two parts thus:

$$\sigma'_a=\sigma'_e+\sigma'_d \qquad 5.$$

The first term σ'_e can be considered as the part of the stress depending on the volumetric strain, i.e.

$$\sigma'_e=f(\epsilon_v) \qquad 6.$$

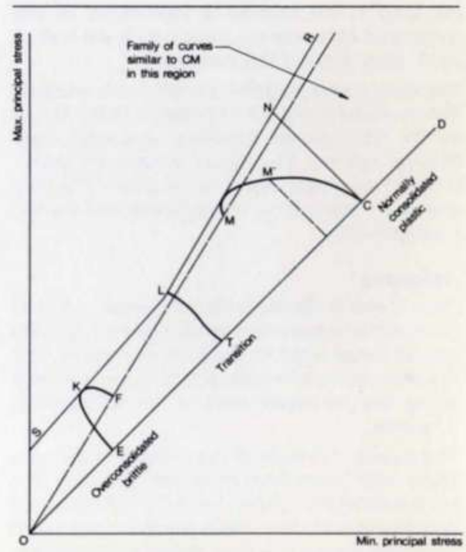


Fig. 1
Idealized effective stress paths

The second term σ'_d is the change of stress due to dilation in overconsolidated clay, or contraction in normally consolidated clay. In the simplest form:

$$\sigma'_d=f(\gamma)$$

If linear, this factor would take the form:

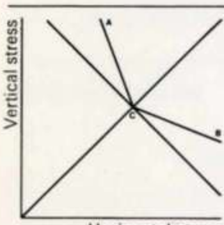
$$\sigma'_d=D\gamma$$

This can be compared with the other linear elastic constants for plane strain, e.g.

$$\sigma'_a=\frac{E}{(1-\nu-2\nu^2)}\ \epsilon_v$$

$$\tau'=\frac{E}{(1+\nu)}\ \gamma$$

If an isotropic clay happened to have a linear 'D' value, then its undrained stress path for vertical and horizontal stresses from a triaxial test would be *CA* and *CB* in Fig. 2 (a). This should not be confused with similar looking stress paths e.g. *CA*, *CB* in Fig. 2(b) obtained from (linear) anisotropic clays as shown by Henkel[1].



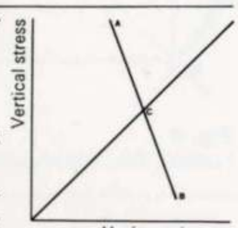


Fig. 2a **Fig. 2b**
Stress paths for a) linear elastic clay and b) linear elastic anisotropic clay

This factor (in a non-linear form) has been used because empirically it seems to be needed to reproduce the experimental stress curves, but, as it is found to apply to a number of quite different stress paths, both overconsolidated and normally consolidated, it appears to be a fundamental property of dilating/contracting materials. In the earlier stages of the work, even before the equations were used in the principal stress form, it was quite easy to obtain axial stress-strain curves which were non-linear. However, the stress paths did not show the divergence from the line *CN* until a failure line, e.g. *LN*, Fig. 1, was reached. With the inclusion of the factor for σ'_d both the overconsolidated and normally consolidated form of the stress paths could be obtained easily.

Because the dilatory stress depends on the level of the mean stress, its equation should be of the form used in Equation 4:

$$\sigma'_d=f(\gamma,\ \sigma'_a) \qquad 7.$$

However it was found that accurate results could be obtained using σ'_e instead of σ'_a so that the equation becomes

$$\sigma'_d=f(\gamma,\ \sigma'_e) \qquad 8.$$

General form of equations

Collecting Equations 1, 3 and 4 together gives the following set of equations:

$$\left.\begin{aligned}\sigma'_a&=f(\epsilon_v)+f(\gamma,\sigma'_a)\\ \tau'&=f(\gamma,\sigma'_a)\\ p&=f(\epsilon_v)\end{aligned}\right\}\ 9(a).$$

It should be possible, in principle, to use Equation 9(a) but it was found that the following form, using Equations 1, 3, 5, 6, 8, was more convenient for programming:

$$\left.\begin{aligned}\sigma'_a&=\sigma'_e+\sigma'_d\\ \sigma'_e&=f(\epsilon_v)\\ \sigma'_d&=f(\gamma,\sigma'_e)\\ \tau'&=f(\gamma,\sigma'_e)\\ p&=f(\epsilon_v)\end{aligned}\right\}\ 9(b).$$

Although the equation was used in this way because it was easier to do so, it seems that this form of the equation more closely represents the real behaviour of the clay.

It appears that Equation 9(b) should be able to reproduce all the physical behaviour of the soil. However, if any other factors are found to have a significant effect, they can be added to the equation.

Functions for general equation

Although in Day[2], the general equations used were of a similar form to Equation 9(a), it was not explicitly stated that the equations used are really only suitable for a restricted range of mean stresses. The form of the functions chosen were simple polynomials.

The only types of functions which were considered in any detail for Equation 9(b) were:

1 Polynomials with integer exponents
2 Polynomials with fractional exponents
3 Exponential curves
4 Multi-linear approximation.

The first three types can be made discontinuous for ease of curve fitting.

Function for deviator effective stress

The work described here was originally directed towards the solution of a problem for a particular excavation in London clay for which the stress-strain curves were available. A curve for this clay is shown in Fig. 3. An integer exponent polynomial appeared to be an obvious choice to fit this curve and it was also a convenient form for programming because of the small amount of data and simple calculation. This form was adopted from the program mentioned in Day[2]. However, when the trials on Weald clay described later were made, it was found that the $\tau'-\gamma$ and $\sigma'_o-\gamma$ curves for this material were difficult to fit with an integer polynomial. The stress-strain curve in Fig. 4 is a typical example for this clay and it is seen that it has a steep section near the origin turning fairly sharply to an asymptote to the horizontal. This type of curve is most easily fitted with a fractional exponent polynomial. The possibility of rewriting the program for real exponents which would embrace both types of polynomials was considered, but eventually it was decided that a multi-linear approximation would be the most suitable. The amount of additional program is not as much as expected and the additional data can be reduced by using common values for the discontinuities on the $\tau'-\gamma$ and $\sigma'_d-\gamma$ curves.

Thus the equation for τ' is:

$$\tau'=\tau'_n+G_n\gamma \qquad 10.$$

Where n depends on γ

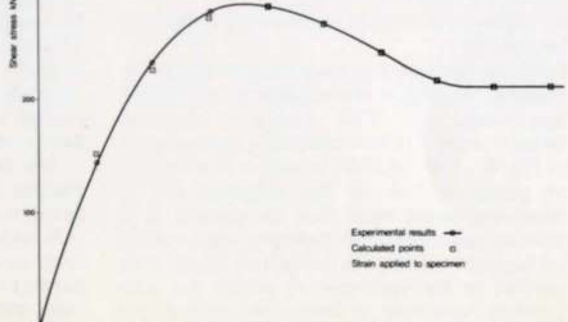


Fig. 3
London clay,
stress strain curve

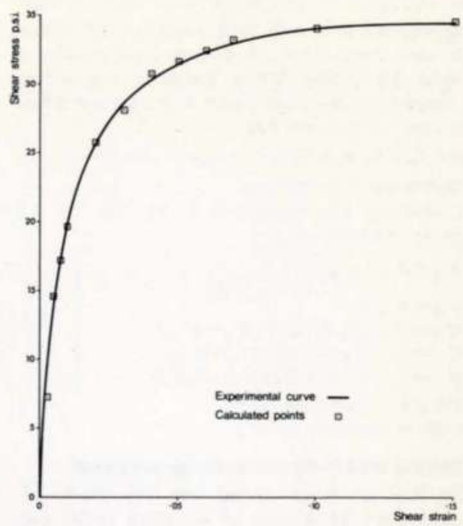


Fig. 4
Weald clay, stress strain curve

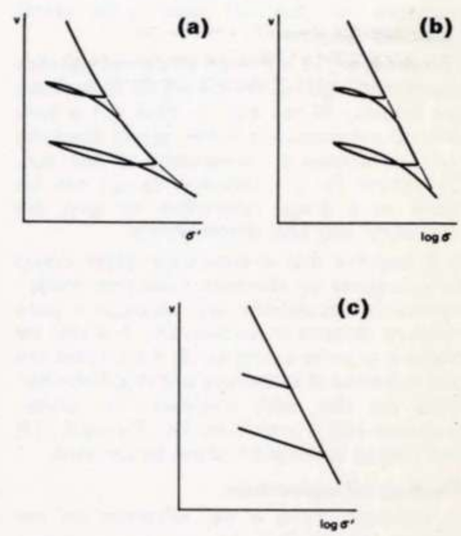


Fig. 5
Volumetric strain – mean stress curves

The advantage of using this form is the ease with which the curves can be fitted.

When a multi-linear form is used for the first part it is more convenient to have the second term in a multi-linear form and to combine the two terms thus:

$$\tau'=G_m(\tau'_n+G_n\gamma)\ (\sigma'_a-\sigma'_m) \qquad 11.$$

Where n depends on γ
and m depends on σ'_e

Function for mean effective stress

A typical graph of specific volume to mean stress is shown in Fig. 5(a), or Fig. 5(b).

This can be idealized to the straight lines in Fig. 5(c), for which the equations are:

$$v=v_o-\beta\, Log_e\,(\sigma'_e/\sigma'_{ao}) \qquad 12.$$

$$\text{so:}\quad \sigma'_e=\sigma'_{ao}e^{[-(v-v_o)/\beta]} \qquad 13.$$

The value of β depends on whether the virgin compression of decompression line is being followed.

In practical calculations it is most likely that the mean stresses will only vary over a short part of the full range of stresses in Fig. 5(a). In this case it may be easier to fit the curve with a polynomial function:

$$\sigma'_e=E_o+E_1\ \epsilon_v+E_2\ \epsilon_v^2\ldots E_n\,\epsilon_v^n \qquad 14.$$

For Equation 8 a polynomial was first used, but again it became apparent that a multi-linear fit would be a better alternative giving

$$\sigma'_d=D_m(\sigma'_n+D_n\gamma)\ (\sigma'_e-\sigma'_m) \qquad 15.$$

When n depends on γ
and m depends on σ'_e

Function for pore pressure

If the clay is saturated, the modulus will approximate to the bulk modulus of water. In case the soil is not fully saturated, provision must be made for a possible non-linear relationship between p and ϵ_v, so a polynomial is used to provide this:

$$p=K_o+K_1\ \epsilon_v+K_2\,\epsilon_v^2\ldots K_n\,\epsilon_v^n \qquad 16.$$

Summary of equations

Collecting the equations 5, 11, 13, 15, 16 gives the following:

$$\left.\begin{aligned}\sigma'_a&=\sigma'_e+\sigma'_d\\ \sigma'_e&=\sigma'_{ao}e^{[-(v-v_o)/\beta]}\\ \sigma'_d&=D_m(\sigma'_n+D_n\gamma)\,(\sigma'_e-\sigma'_m)\\ \tau'&=G_m(\tau'_n+G_n\gamma)\,(\sigma'_a-\sigma'_m)\\ p&=K_o+K_1\,\epsilon_v+K_2\,\epsilon_v^2\ldots K_nK_v^n\end{aligned}\right\}\quad 17.$$

Where n depends on γ
and m depends on σ'_e

Initial and long term deformations

In the program based on Equation 17, alternative behaviour of the pore water can be specified:

1 There is no movement of pore waters between elements.

2 There is either no change of pore water pressures or specified final pore water pressures are given.

The effect of (1) is that an undrained calculation is made, giving the immediate deflections (or failures) of the soil. In case (2) a fully drained calculation is made, giving the long term consolidation movements of the soil. Calculation for (1) followed by (2) can be made as a stage calculation to give the immediate and final deformations.

It is possible that intermediate stages could be calculated by alternating between stress-formation calculations and seepage – pore pressure dissipation calculations. It would be possible to write a program to do this, but the two extremes of immediate and final deformations are the most important for design purposes and it is unlikely that the cost of a multi-stage calculation could be justified.

Testing of equations

In principle, there is no restriction on the number of coefficients or discontinuities which may be incorporated in the coefficients of Equation 17. From examination of Fig. 1, it would seem reasonable to presume that there is a discontinuity in the curves along the line *CM* as this line separates the overconsolidated zone from the zone where further normal consolidation can occur. Thus, it is necessary to provide coefficients to fit the curves at *CM* and *EK*. For the calculations on Weald clay described later, two additional sets of coefficients for intermediate points were required.

A program was written to calculate the stresses in a single, uniform stress, rectangular element based on Equation 17 and having sufficient coefficients to cover all the curves found in Fig. 1, from those near *O* to those beyond *C*.

The purpose of this program is first to test the validity of Equation 9(b) and second to be used as a trial program for deriving coefficients for multi-element programs. The single element can be considered to represent any of the elements of the multi-element calculation when it is subjected to arbitrary strain or stresses so that its behaviour can be studied in isolation. In addition, the single element can be taken as a numerical representation of specimens of the soil and made to follow numerically the imposed stresses or strains of testing apparatus.

It should be emphasized that the single element program was written purely for convenience as it could be easily and quickly altered to test modifications and variations of the equations. The multi-element program for one form of the equation was written first but, because the form chosen for the equations was not satisfactory and it was not easy to produce numbers of stress-strain curves, the single element program was written to establish the best form of the equations. The method of solution is identical in both programs and the multi-element program (with modifications to include additional constants) could reproduce the results quoted later. When the right form of the equations had been developed the multi-element program was modified to incorporate the changes.

This figure represents an experiment in which the clay is consolidated at a pressure of 120 psi. Triaxial tests at initial stresses of 70.5, 60, 44.5, 30, 15 and 10 psi give the stress paths shown in the figure. The strains have been noted adjacent to the relevant stress point. It can be seen that at confining pressures, which are low relative to 120 psi, the stress paths show typical overconsolidation dilatory shapes while at pressures approaching 120 psi the curves change to approach the normal consolidation curves.

Again the exponential coefficients for the virgin consolidation and overconsolidation lines were not available from these tests so the values quoted in Table 6.1 in Schofield and Wroth[3] have been used thus:

$\kappa=0.035 \quad \lambda=0.093$

Values for the coefficients for Equation 17 were derived and then the numerical test specimen subjected to a number of tests. In all these tests the specimen is initially at confining stress and effective stresses of 10 psi with zero pore pressure, i.e. the specimen is at point *E* in Fig. 8.

1 Undrained volumetric strain

By applying equal vertical and horizontal strains to the plane strain specimen, changes in volumetric strain are produced. From test calculations the following table was obtained:

Vol. strain	*Effective stress*	*Pore pressure*
0.0	10.00	0.0
0.0002	10.08	65.4
0.0004	10.16	130.8
0.0006	10.24	196.2
0.0008	10.33	261.6
0.001	10.41	327.0

It can be seen that very little effective stress change occurs, practically the whole of the total stress arising from the pore pressure.

2 Drained volumetric test

In this case the pore pressure is dissipated so the whole of the total stress is carried by the effective stresses so that from this test the curve in Fig. 9 is obtained. It can be seen that there is a discontinuity in the curve at the consolidation pressure of 120 psi.

3 Undrained biaxial test with constant horizontal stress

With a constant horizontal total stress of 10 psi, vertical strains are applied which produce the points by the experimental curve *EK* in Fig. 8.

A plot of the shear stress to strain for this calculation is shown in Fig. 4.

4 Drained volumetric followed by undrained biaxial test

From the first test it was seen that if volumetric stresses or strains are applied to undrained specimens, very little change of effective stress occurs. If initial conditions are at point *E* in Fig. 8 – then in order to obtain stress paths at points further up the diagonal *OD*, a confining stress must first be applied to a drained specimen. For example, equal initial vertical and horizontal stresses of 30 psi were applied to the specimen in which the pore pressure was held at zero. The vertical and horizontal effective stresses change from the initial value of 10 psi to 30 psi with the strains

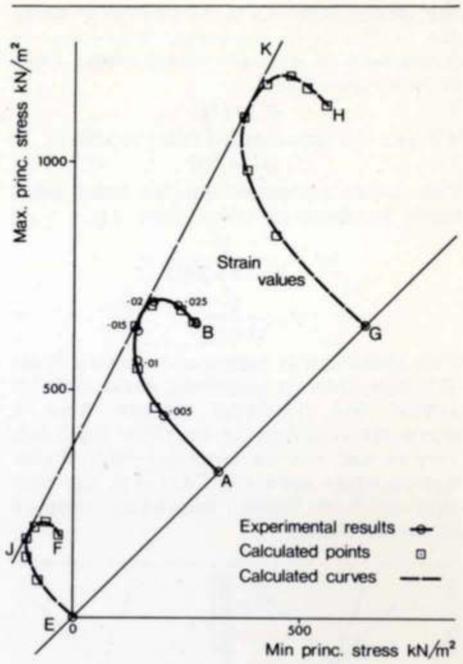


Fig. 6
London clay, stress paths

following Fig. 9. With a total horizontal stress held at 30 psi, increments of total vertical strain were applied to an undrained specimen. The resulting effective stresses calculated are plotted adjacent to curve *FG* in Fig. 8.

Although a single element may be able to reproduce the experimental stresses and strains, it does not necessarily follow that the multi-element program will give the correct result when it uses the same coefficients for a design calculation. However, the converse is certainly true, i.e. if the experimental results cannot be reproduced, then it is not reasonable to expect accurate results for design calculations. This is the reason the following work has been done to establish the adequacy of the equations for a single element.

This program was then used to calculate the stresses and strains to compare with two sets of experimental data for:

1 Brittle over-consolidated clay

2 Clay with stress path in the whole range between over-consolidated and normally consolidated.

1 *Brittle over-consolidated clay*

From test work carried out to obtain data for an excavation in London clay, the curve *AB* shown in Fig. 6 for the soil at 18 m depth can be derived. Within the limits of the tests it was found that the strength and stiffness of the clay was directly proportional to the confining pressure, so it is assumed for this trial calculation that the shape of the stress path at any point on the diagonal is geometrically similar to *AB* with the maximum shear fixed by the line *JK*. The maximum mean stress for the strata in which the excavation would be made was about 600 kN/m². For this clay the normal consolidation pressure is of the order of 3500 kN/m². This means that the point corresponding to point *C* in Fig. 1 is right off the diagram in Fig. 6, so that for this case, no data are available at a point equivalent to point *C*. Fictitious data were created for this point which would not affect the behaviour in the region of the stress paths in the figure. The variation of stresses with volume change was not obtained in these tests so the κ value for London clay (0.062) quoted in Schofield and Wroth[3] (Table 16.) was used. After some trials a set of coefficients which seemed satisfactory were found and they were then tested as follows. With an initial effective vertical and horizontal stress 335 kN/m² and zero pore pressure (point *A*), increments of vertical strain were applied and they gave the calculated values plotted adjacent to the experimental line *AB*. The calculation was repeated with horizontal strains and this test gave the same principal stresses, as this clay is assumed to be isotropic.

Repeating the calculations with initial effective stresses of 670 and zero kN/m² gave the lines *GH* and *EF* in Fig. 6 respectively. For the latter cases, the discrepancy from the experimental curves would be proportional to that of the first curve.

For the calculations with initial stress of 335 kN/m² the stress-strain curve is plotted in Fig. 3 for the case where strains are applied. When stresses are applied the curve in Fig. 7 was obtained and it is seen that after the peak stress is reached, the element collapses with indefinite increase of strain.

2 *Over consolidated – normally consolidated clay*

Part of the experimental work for Henkel[4] consisted of consolidating reconstituted Weald clay at a fixed stress for a period of time and then testing it at various stresses. From these experiments, David Henkel has kindly provided curves from which the experimental curves in Fig. 8 have been

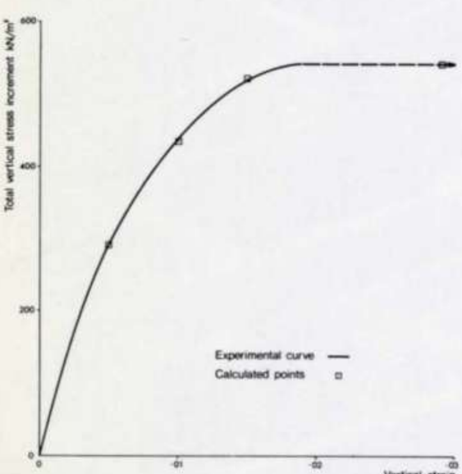


Fig. 7
London clay, stress strain curve.
Stress applied

derived. The pressures are given in imperial units to correspond with the original.

These calculations were repeated for each of the curves which started with equal effective stresses of 15, 45, 60, 70 and 120 psi and the calculated values are plotted in Fig. 8.

There appeared to be an anomaly after a strain of 0.1 in the experimental curve with an initial vertical stress of 30 psi (curve *FG*). The part of the curve after 0.1 strain had to be ignored as it affected the fit of the remaining curve if it were included. Apart from this, it can be seen that for many of the points, the calculated value is within 1 psi of the experimental value. (The calculated point is not shown if it falls within the circle of radius 1 psi marking the experimental points). The greatest error is in the vertical stress at 0.0025 strain in the curve starting at an initial vertical stress of 60 psi.

Equations for design calculations

In a practical calculation for excavation of earthworks, it is unlikely that the variation of stresses in any of the soil strata will be more than a small range of average stresses covered in Fig. 1. Thus for an excavation in over-consolidated clay the average stresses for a soil layer, at say 15 m is of the order of 300 kN/m² and will never approach the order of the consolidation pressure, say 3000 kN/m², during a calculation, so the number of coefficients used in a multi-element calculation can be reduced. It should be noted that there is no restriction on the relative position of the values in the stress range to the stress value at *E, T,* and *C* in Fig. 1 for each of the soils making up the strata in a calculation. The amount of data available for a calculation is generally limited by the amount of testing which can be done and a small number of coefficients should be adequate.

Comments and conclusions

Because all the physical effects of volume and shear strain and dilation which can be envisaged are being reproduced numerically in the equations proposed, there is no difficulty in reproducing the general form of experimental curves. The degree of accuracy of fitting actual experimental data then reduces to a matter of the amount of data which will be provided for a program. In principle, any set of curves can be fitted to any degree of accuracy required. In practice, the fit

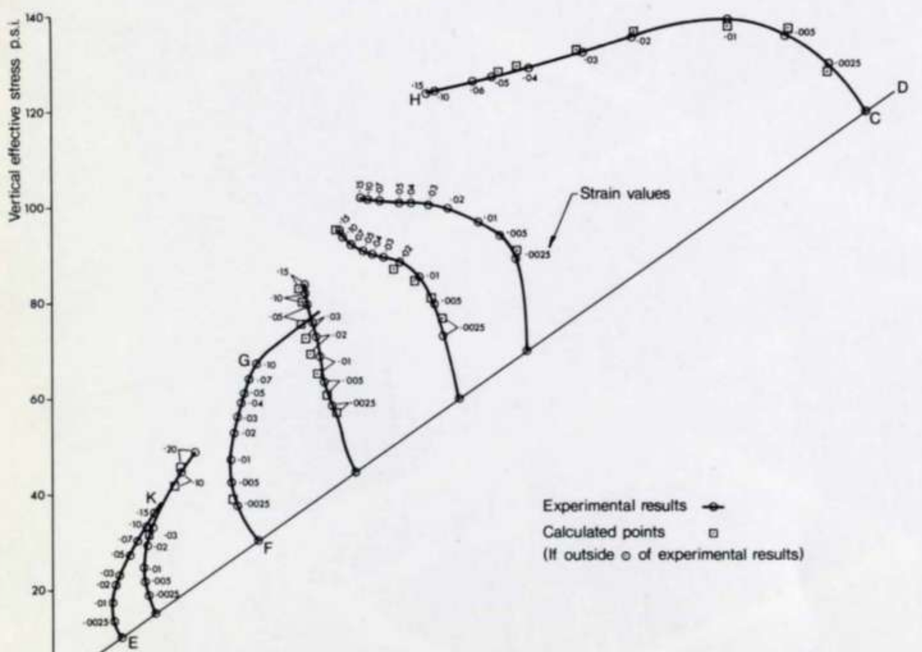


Fig. 8
Weald clay, stress paths

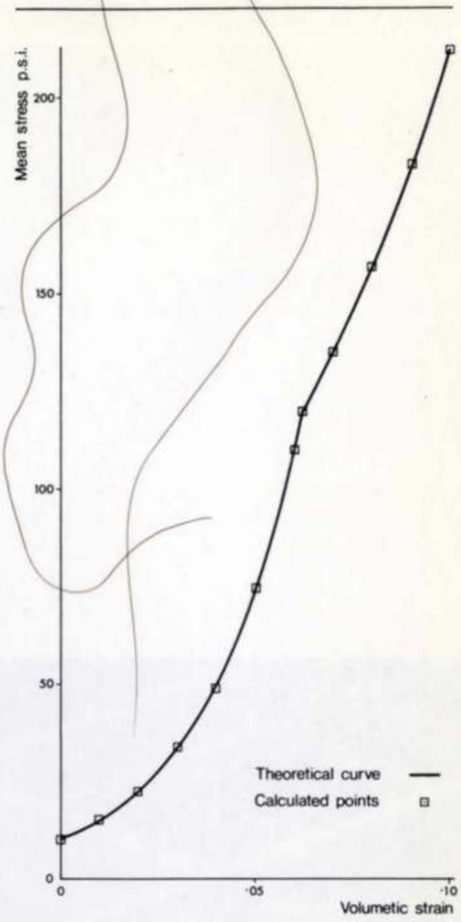


Fig. 9
Weald clay, mean stress – volumetric strain curve

obtained in Figs. 5 and 8 should be adequate, especially as the fit to the mean experimental curve can be closer than the scatter of the experimental curves.

From an examination of typical curves for sands, it appears that Equation 9(b) would be adequate for their analysis, but as the experimental data are obtained in a different way, it may be more convenient to revise the form of the equations. For jointed rocks an additional term relating the shear and normal stresses on the joint planes would be required.

The accuracy of reproduction of the experimental data considered above indicates that the proposed constitutive equations should be satisfactory for calculating the non-linear stress-strain behaviour for uniform stress elements in a finite element analysis, for soil and other non-linear materials, and in particular the effects of dilation are being adequately represented.

Acknowledgements

It has been extremely useful to be able to discuss various points with others, and I am especially grateful for the help I have had from Poul Beckmann, Edmund Booth, John Blanchard, David Henkel and Brian Simpson.

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